LONGWALL DEMONSTRATION IN WESTERN CANADIAN COALS

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For Canada Centre for Mineral
and Energy Technology

14 January 1983

APPENDIX 3-1

AN ASSESSMENT OF THE GEOLOGY OF THE PROPOSED LONGWALL AREA OF NO. 9 MINE

NORWEST RESOURCE CONSULTANTS

AN ASSESSMENT OF THE GEOLOGY
OF THE PROPOSED LONGWALL AREA
OF NO. 9 MINE

THE ASSOCIATION OF
PROFESSIONAL ENGINEERS,
GEOLOGISTS and GEOPHYSICISTS
OF ALBERTA
PERMIT NUMBER
P 914
NORWEST RESOURCE
CONSULTANTS LTD.

Prepared For: McIntyre Mines
Prepared By: Norwest Resource Consultants Ltd.

September 1982
Report No. 82-235/1

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1. INTRODUCTION

The following report has been prepared at the request of Mr. Howard Owen of McIntyre Mines Ltd. The purpose of the study is to provide an independent assessment of the geological data that is available for the proposed longwall area and to determine the suitability of the geology of that region for longwall mining. An analysis of potential geological hazards that could affect longwall mining has also been requested.

The evaluation required that most of the study be conducted at the mine site so that drill core, the surface exposure of the strata, and the underground and surface workings could be examined. In addition, all the raw exploration data is available at the site, as are the geologists who compiled the geological data package. Although the work schedule of the McIntyre personnel is a particularly heavy one, the fullest cooperation was provided to the writer and several meaningful and constructive discussions took place. The writer wishes to thank those employees of McIntyre Mines and emphasizes that their assistance has been of major benefit to the completion of this report.

2. EXPLORATION ACTIVITY TO DATE

Most of the exploration work done to date involved drilling by various methods. Drilling is the most effective technique because the principal objectives of exploration are to map of the seam horizons in the subsurface, to establish the lateral continuity of the seam and its in-seam sediments, to determine the presence of small-scale faults at the seam level, to assess the variability of roof and floor strata, and to test for the presence of relatively soft zones within the coal. Thus, the extensive use of drilling in this area is essential to successfully understanding the geology as it relates to an underground mining situation.

Some surface mapping has been conducted, and there is still the scope to continue this type of exploration work. However, it does not appear that an expanded program of surface mapping would be an essential requirement for a decision to proceed with development. Detailed examination of the existing surface exposures of the seams, particularly No. 4 Seam may be beneficial in developing an understanding of the distribution of soft, sheared coal layers. This work could be incorporated with work already undertaken to evaluate this geological condition from borehole data.

The examination of the surface expression of faults would be useful in developing an understanding of the angular relationships that exist between the faults and bedding in the various structural environments and in blocks of strata having various orientations. This information would facilitate the projection of faults intersected in the various drill holes to the elevations of the mineable seams.

The potential for the use of other exploration techniques has also been considered. These techniques might include air photograph and satellite imagery interpretation, adit drivage, various techniques of surface remote sensing by geophysical methods, and borehole seismic exploration. Interpretation of air photographs was performed at an early stage on this property, and does not appear to have the potential to enhance the results of the extensive subsurface mapping that has already been conducted. Adit drivage to establish coal quality, and detailed strucutural and stratigraphic situations, is more than fully compensated for by the amount of underground drivage that has already been completed. Surface geophysical techniques do not appear to have the potential to play a significant role in the exploration of this area because they have not yet been demonstrated to produce results with a consistent and acceptable level of reliability in mountain coal situations. Surface geophysical techniques are normally applied in situations of far less advanced levels of exploration. Borehole seismic as an in-seam exploration technique has yet to be fully developed and so should not be undertaken at this time. Other techiques might include the application of sedimentary depositional modelling, but the extensive use of drilling has already compensated for any benefit that might be gained from this exploration method.

Consequently, it is the opinion of the writer, that the exploration programs conducted to date within the area planned for development are more than adequate to achieve an acceptable level of confidence in the knowledge of the geology and to allow the appropriate geological decisions affecting development to be made.

It should be noted that it is common, in mountain situations, for clustering of the drill holes to occur as a result of difficulties of access. In the case of the No. 9 Mine Area, clustering does not occur and in general there is an even distribution of the drill holes with a slight emphasis on more closely spaced drilling in the region of earlier planned mining activities. This is a desireable situation since the requirement for additional drilling can be determined as the mining advances.

There are fewer cored drill holes than is normal in an area proposed for mining, but two factors compensate for this possible shortage: the area with the highest requirement for coring has already been explored by extensive underground workings, and the most recent drill holes have been subject to full programs of modern geophysical logging. These logs can provide almost as much information as core and are often more reliable. Features such as the detailed variations of roof and floor lithologies, seam elevations, seam thicknesses, thickness of in seam sediments, lateral continuity of various sedimentary features, and the structural repetition of strata can all be more accurately evaluated using modern geophysical borehole logging techniques. It is the opinion of this writer that some of the available logs will also allow a better understanding of the distribution of sheared coal horizons within coal seams than could be achieved by core drilling.

The drill hole spacing is quite close, with the along-strike spacing being about 150 metres and the cross-strike spacing being about 100 metres. This density is unusually high and, in fact, exceeds the density of drilling for almost all the potential production situation's that have been experienced by the writer. In only one situation has the density of drilling been greater than that found in this project area and that occurs on the Corbin Property of Crowsnest Resources. The latter situation is an especially structurally complex one unlike the subject area.

Exploration of the target development area has been conducted in several different years from 1969 through 1981. This period has been one of the most significant for the advancement of geological exploration techniques, especially for drilling and borehole geophysics. The writer has personal experience of most of the drilling companies used on the various exploration programs and it is apparent that good equipment and personnel have been used. This is an extremely important consideration because the reliability of the interpretations drawn from the sampling is dependent not only on the drilling equipment available but also on the specific people conducting that sampling. In the use of borehole geophysics as a sampling technique, McIntyre Mines have employed one of the foremost contractors for this service available in the world. The writer is fully qualified to comment in this regard having recently completed a text for the Coal Mining Research Centre which included in part an evaluation of borehole geophysical logging for coal on an international basis. Four companies supply particularly reliable borehole geophysical logs and the contractor employed by McIntyre is one of these. In fact, that contractor conducts a higher level of testing to determine the relationship between the log trace and the strata being measured than any other company.

The present status of the proposed development area is that it has been fully explored for geological purposes to allow a development decision to be made. Specific detailed examination of existing coal seams and fault exposures may be of assistance in the interpretation of data but is not a critical item.

3. THE INTERPRETATION OF DATA

The interpretation of data involves two steps:

- the extraction of data from the sample information available, and
- the plotting of that data in the form of maps and cross- sections.

Although the drill hole sampling has been conducted in various programs since 1969, all of the information has been recently reinterpreted for the purposes of preparing the current set of geological maps. The quality of the raw data of course varies from year to year with the better quality data being available in more recent years, but that information has been interpreted using the most modern techniques which have developed based on the experience gained during many of these exploration years. In fact, the most recent geophysical logs have been suitable to allow use of perhaps the most modern method of data extraction, a technique yet to be published. The most critical items that must be determined in the preparation of this extracted data are: the collar location of drill holes, the location of seam intersections within those drill holes, the thickness of seams and in-seam sediments, the lithology of various stratigraphic units, accurate seam elevations, and the identification of the nature of in-seam materials.

The collar locations of all drill holes are surveyed. Although this may not seem significant, in many exploration programs the drill holes remain unsurveyed for a long period of time and often it is difficult to determine the location of early holes. Borehole orientation logs have been produced for the more recent drill holes since appropriate logging techniques have become available. For older holes, the orientation is assumed to be vertical or the collar orientation, and the geological uncertainy of seam intersection locations on this basis has been brought to the attention of the writer by the geologists of McIntrye Mines. The writer does not believe that this is a major factor affecting the interpretation of data, because surrounding drill holes tend to confirm the current interpretation since no anomalous trends which might result from this uncertainty of orientation have been found. It should be noted that many of the projects currently being considered for development suffer from a similar geological uncertainty, and this is by no means an unusual situation.

The seam thicknesses for the geophysical log have been determined by the writer through a rapid cross checking method using density and gamma-ray logs. This technique tends to be somewhat conservative but provides a good comparison with the thicknesses determined by the McIntrye staff. The thicknesses for No. 10 Seam and No. 4 Seam determined by the writer are on average 1 foot less in thickness for the drill holes prior to and including 1976, and are about 6 inches less for the subsequent drill holes. Only one significantly anomalous value was found in the interpretation of one of the No. 10 Seam sections, and adjustments to the No. 10 Seam maps have already been prepared. Although there are general differences between the results of the writer and those of McIntrye staff, they may result from the writer's use of a rapid technique, and it is not known whether a closer correlation would be achieved by a more time-consuming and accurate method. The differences are best described as one geologist's opinion compared with another when different data extraction methods are used. In the cases of both No. 10 Seam and No. 4 Seam, it does not appear that these differences, if real, are of any major consequence to the development of either seam and this is especially apparent in the case of No. 4 Seam. Wherever available, the intersection of the drill hole at the seam level has been plotted rather than the collar location, and the appropriate adjustment to the elevation of the seam at that point has been made. Depths to the seam as shown by the geophysical logs were determined by the writer and compared with results displayed on the map. Only one significant difference was located, and that situation is commented on below The writer also examined surface exposure of No. 10 and No. 4 Seam as well as subsurface exposures of No. 4 Seam and compared the nature of those materials with information illustrated on the various tables. The only differences detected are commented upon below. Overall, the techniques used to extract and plot data are modern and accurate ones which provide as reliable an understanding of the location of data points and the nature of materials at those points as is possible to achieve.

The second stage of data interpretation involves the plotting of the extracted data onto various maps, and the interpretation of results by contouring. In addition to illustrated sectional data, two types of contour maps are available: isopach maps and structure contour maps. The technique of contouring structure contour maps differs from that applied to the isopach maps because it is necessary to accurately represent gradual changes of the curvature of strata. This has been done in a very thorough manner for the structure contour maps which are available for No. 10 and No. 4 Seams. Only one significant change was necessary to these maps in the opinion of the writer, and that change is due to a rather difficult correlation problem existing between the drill hole at the edge of the property (Hole 8129) and those more centrally located. The writer was advised of this difficulty by the McIntrye geologists, and a joint investigation of that hole by the writer and McIntyre geologists identified the miscorrelation. The geologists immediately undertook to make the necessary modifications to the maps. It should be noted that this change demonstrates a higher level of continuity of the geological structures through the area than was formerly apparent.

The isopach maps have been prepared on a vertical thickness basis, which makes the determination of reserves considerably more simple. Contouring has been done according to a standard technique involving the inclusion of single data points between drill holes, but an additional modification has been added to accomodate tectonic thickening of strata in fold axial regions. The technique is commonly employed in foothills and mountain-type terrain. The writer examined all of the maps in detail and found few instances of arithmetic errors in the data plotted on the maps and these changes have been subsequently incorporated in a revised version of those maps. It should be noted that these changes indicated a higher level of continuity of the various stratigraphic units than was displayed before.

The No. 10 Seam isopach map however, deserves comment. Some anomalous values appear on this map, creating the impression of a relatively high degree of seam thickness variation. However the data points which suggest this variation were obtained from drill holes without geophysical logs and for which only the driller's record is available. It is probable that the section recorded by the drillers does not include the upper thermal coal section and relates only to the higher quality, lower metallurgical coal section. This matter was discussed with the McIntrye geologists responsible for preparing the map and their opinion is similar. It is worth noting therefore, that these are data points of lower reliability, and there is no way of accurately determining the nature of No. 10 Seam at those locations. If this judgement is correct, then the isopach map of No. 10 Seam should show a much higher level of continuity than is currently displayed. The stratigraphic correlation change noted previously in affecting Hole 8129 affects the isopach maps as well as the structure contours and appropriate modifications have already been made.

In an overall sense, good techniques have been use in data preparation, compilation, and contouring to prepare maps that are representative of the geology of the area. All of the points of difference noted in the foregoing discussion have been incorporated into the current set of geological maps. A high level of confidence can be placed on the reliability of the geological structure and the variation of vertical thickness of the various stratigraphic units displayed on those maps.

4. SEDIMENTARY VARIATION

The borehole geophysical logs provide the best record of variations of the strata throughout the planned development area. These logs show that the outer seam sediments vary to only a minor extent and that the lithologies of the roof and floor sediments of the seams are consistent through large areas. The seam signatures of No. 10 and No. 4 Seam show high level of lateral continuity. The greatest variation occurs in the upper part of No. 10 Seam. This seam is characterized by an unsheared lower metallurgical coal section, overlain by a rock parting and then a very heavily sheared upper thermal coal section. In some instances the upper rock parting as well as the overlying coal is intensely sheared and in that situation argillaceous material from the rock parting is included with the sheared coal. Similarly, sheared coal is incorporated with the rock material where the rock band has been intensely disturbed. These tectonic affects are reflected in variations on the geophysical logs from hole to hole where the conditions change. The presence of the rock band in No. 10 Seam will be inconsistent throughout this area, not as a sedimentary unit but rather as a varying, tectonically disturbed zone sometimes incorporated within the overlying coal and in other instances separated from it.

A brief examination was also made of another seam, No. 11 Seam, which could have potential to be mined underground. The thickness of this seam was noted to vary from essentially zero upwards. It appeared, from this examination, that variations of the internal seam characteristics were greater for No. 11 Seam than for either No. 10 or No. 4 Seams, and therefore a significant evaluation of No. 11 Seam did not appear warranted.

The characteristics of No. 4 Seam are surprisingly consistent throughout the area. This seam consists of three coal splits separated by two small rock bands. The ash content appears, from the geophysical logs, to increase downward through the seam, and the upper rock band is either very thin or consists of a thin zone of high specific gravity coal. The lower band is always present in the seam sections and consists of carboneous mudstone which is always intensely sheared. Intense shearing of this band throughout the subject area can be predicted on the basis of the level of tectonic deformation, and stresses and failure occuring within the seam. This unit is most important in consideration of the location and thickness of the zone of shearing in No. 4 Seam, discussed in detail in Section 7 of this report.

The information available indicates that the strata within the project area, and particularly the coal seams, show a very small amount of stratigraphic variation. No significant variations of lithology or thickness between the drill holes is anticipated.

5. STRUCTURAL VARIATIONS

The structure contour maps and cross sections show that the structure of the area consists of relatively gentle and broad easterly dipping sections of strata separated by units with a somewhat steeper westerly dip. The fold axial regions separating these blocks of strata are relatively broadly folded in a more or less concentric orientation and have an insignificant plunge. No large-scale faults have been detected in the area, and the exploration data available at the present time essentially excludes the possibility of such features being present as a substitute for the interpretation of the presence of a fold. Some small-scale thrust faults are present as described in Section 6. The structure presented on the maps appears to be an accurate representation of the structural geology of the area. The interpretation does not suggest the presence of any anomalous areas which could pose a particular hazard for mining as a result of excessive deformation, excessive stress build-up, or incorrect interpretation.

6. SMALL-SCALE FAULTING

Some small-scale thrust faults occur within the proposed development area. Several small features of this type have been encountered in the existing underground workings where they have a displacement of about 1.5 feet and a separation of about 200 feet. The structures which have been encountered to date have had no noticeable impact on mine development. They are located on the axis of an anticline at the point of a major change of the axial trend. Since this location is characterized by a particularly high level of tectonic disturbance, it is anticipated that this is the maximum frequency of occurence for these features.

The written core descriptions for holes for 8122, 8123 and 8129 suggest that the strata have been subjected to an intensely high level of structural disturbance, but examination of cores and core photographs indicates that is really not the case. In fact, in the four core holes that have been drilled, a total of four zones can be found which may have tectonic significance. Geophysical logs indicate that two of the zones are of no consequence whatsoever, and the other two represent small-scale thrust faults. The enclosed plans No. 1 and 2 are detailed analyses of these structures. The geophysical logs shown are included for illustrative purposes only; additional geophysical logs were used to established the detailed correlation. The faults in holes 22 and 23 have maximum possible displacements of about 2 feet and about 7 feet, respectively. The actual displacement where the faults intersect the seam may be smaller than this, since it is not possible, from the geophysical logs, to take into consideration any variation of the dip of the strata on the other side of the fault plane. These structures are expected to follow the regional and local structural trend, and although they may dip in an easterly direction, it is more likely that they have a westerly trend. Projection of the fault to the of the various seams is difficult since there appears to be a single intersection in the drill holes. However, in similar situations the overall angular relationship between the fault plane and bedding is about 10^0 to 15^0, and projection of these structures from the point of intersection in the drill hole to the seam elevations using this relationship would provide a reasonable estimate.

It is not possible to accurately predict a frequency, or indeed a location, for features of this type, but it is expected that they will have a wider spacing than those already encountered in the underground development drivage. Areas of higher potential frequency of these features are illustrated on the hazard map (Map No. 7). This map implies a potentially higher level of frequency in the region of fold axes and in the more intensely deformed strata which appear to be present on the westerly dipping limbs.

7. THE NO. 4 SEAM SHEARED COAL ZONE

Drill cores, and surface and underground exposures of No. 4 Seam show the presence of a zone of very intense shearing which contains exceptionally soft coal which will have a major impact on mining. The investigation found that the material within this zone is quite consistent in its physical appearance; the sheared material consists of small slivers of coal, and sometimes rock, slickenedsided on all surfaces, with an absence of any bonding material such as calcite or silica. The sheared zone appears to have extremely low cohesive strength and displays many of the characteristics of very poorly lithified sediments.

The investigations also found that the thickness and distribution of the sheared material are quite variable in different structural domains. The sheared zone appears to be present to some extent in all examples of Seam No. 4, with an increasing thickness as the level of tectonic deformation increases. The sheared zone always includes the rock band in the lower two-thirds of the seam. This can be predicted on the basis of the thickness of the units and the contrast of mechanical properties under conditions of relatively high stress. This type of deformation is a plastic mechanism unlike the semi-brittle deformation that has occured through much of the surrounding strata. Coal seams are normally affected by this type of deformation before any of the surrounding rocks due to the constrast in mechanical properties, the presence of thin rock bands within the seams, and the thin nature of the lamination of the coal itself. Failure of this type occurs as a result of the overall shortening of the sedimentary pile during a phase of tectonic activity, and it achieves its maximum development where conditions of anomalously high tectonic stresses are encountered.

As a result, certain predictions can be made concerning the distribution of the sheared zone, and possibly its thickness, through the proposed development area. The sheared zone will have a minimum thickness in the gently east-dipping limbs of the folded sequence of strata, and a maximum development in fold axial regions or adjacent to thrust faults. An intermediate level of development can be predicted for the relatively steeper westerly dipping limbs where the amount of rotation of the strata, and bedding-plane movement, will almost certainly be greater. In addition, it is possible to predict that the shearing is most likely to have occurred adjacent to and including any thin rock partings where the mechanical contrast between thin-bedded strata is the greatest. These theoretical considerations suggest that the sheared zone will be thinnest in the easterly dipping strata, that it will become progressively thicker or develop into several zones of shearing of the coal on the steeper westerly dipping limbs, and that it will achieve its greatest thickness adjacent to thrust faults or in fold axial regions.

Theoretical considerations also suggest that it should be possible to detect these zones and determine their thickness from appropriate borehole geophysical logs. Through experience, it has been observed on various properties along the Rocky Mountain Foothills that sheared zones of this type are commonly water-bearing, and wet coal consistently displays relatively low resistivity values on focused resistivity logs. In addition, these soft zones are normally subject to caving during drilling and so may be recorded on the caliper log. Density logs also respond where changes in porosity occur within the zone of shearing. Thus, comparison of anomalous responses on focused resistivity, caliper, density, and gamma-ray logs can be used to detect the extent of development of the sheared zone in a coal section. Map No. 3 is an attempt at a preliminary analysis of this type for the proposed development area. The three example logs on the left of the plan are located within the easterly dipping strata, while the central and right-hand examples are taken from holes drilled in the steeper westerly dipping strata and the fold axial region, respectively. It appears at this stage that the theoretical predictions discussed above are supported by the actual development of tectonically sheared coal in each of those drill holes.

The three drill holes on the left of the illustration are typical of the conditions that are currently being mined in the underground development work for the proposed development area. At the limit of this development work, a complete seam section has been exposed which indicates that mining conditions should not be especially difficult from the point of view of shearing and soft coal. This will be the case as long as a sufficiently thick layer of hard coal is left to protect the sheared coal layer at depth. Because there is no bonding material between each fragment of coal, and because the sheared coal layer and individual coal fragments already display high levels of polishing, this material would be subject to flow under any direct loading and would not withstand the abrasive action of mining equipment traveling over it. The thickness of hard coal required above the sheared zone to restrain flow of the sheared material will have to be determined from geotechnical parameters initially, and through mining experience at a later date. A recommendation to continuously evaluate the location and thickness of the sheared layer is presented in Section 9 of this report.

8. SEAM NO. 4 HAZARD ANALYSIS

The principle concerns in a hazard analysis for No. 4 Seam are the possibility of encountering thrust faults on a small or large scale, and the potential for increasing thickness of zones of shearing within in the seam. The data available at present suggests that significant variations of the thickness and nature of the lithological units are not a major concern and risks caused by variations in data point spacing have been overcome by a generally equal distribution of data. Therefore the following factors have been included in the hazard analysis:

- the drill hole condition recorded by the drillers, geophysical logging engineers, etc.;
- the extent to which No. 4 Seam shows caving;and
- the degree of variation of orientation of the beds, and the dip as shown on the structure contour maps.

Maps No. 4, 5 and 6 show each of these parameters contoured separately and Map No. 7 is a composite contour map of these three factors. The contour values shown on the maps are subjective and have been selected on a purely arbitrary basis, but they are intended to reflect the order of importance of changes within each parameter. Consequently, higher contour values on the hazard analysis map are intended to show increasing possibilities for both small- and large-scale thrust faulting as well as for a thicker zone of in-seam shearing. There is no evidence to suggest the presence of major thrust faults, and so it is expected that, if thrust faulting is found it will manifest itself as small- scale features. It should be noted that linear trends of the contour lines on the map reflect the trends of the fold axes and of relatively abrupt changes of dip of a monoclinal nature from the structure contour maps. It should also be noted that the area of planned development has, overall, the lowest contour values and that significantly higher values occur principally in the fringing areas.

9. RECOMMENDATIONS FOR FUTURE WORK

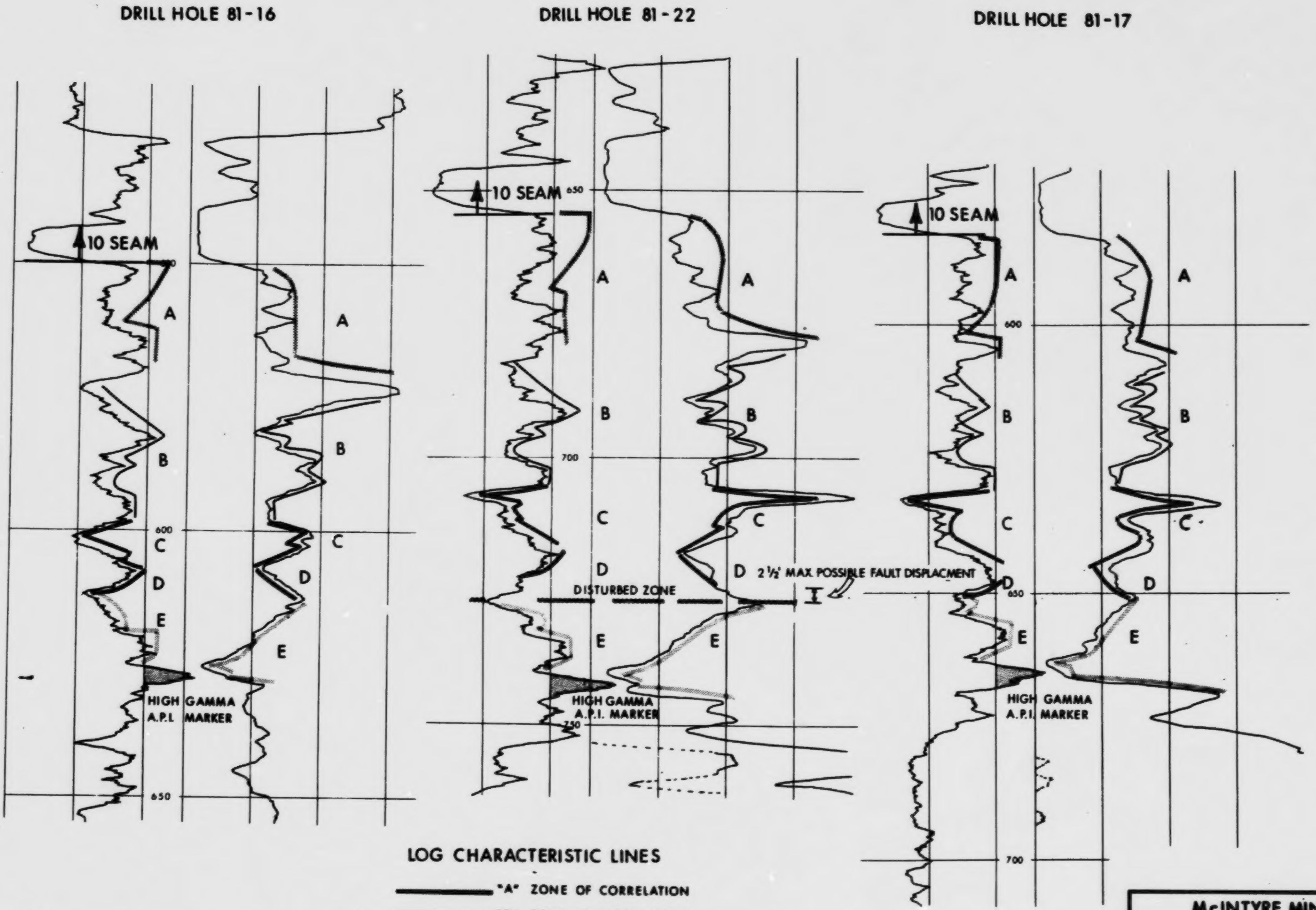
Recommedations have already been made in Section 2 concerning the detailed mapping of seam exposures and faults as an aid to structural interpretations; however, that work is not of a critical nature. The distribution and thickness of the sheared zone in No. 4 Seam, on the other hand, is most important and every effort should be made to document the thickness and variation of thickness of this unit. This would be a difficult problem to resolve by surface exploration drilling since the holes would have to be produced at a relatively wide spacing. The work would best be achieved underground on a pillar-by-pillar basis. Norwest suggests that a drilling unit be mounted on the bolter and that holes be drilled into the floor of the seam as frequently as is deemed necessary. The identification of the depth and thickness of this unit could be made simply by the penetration rates of the drill steel into the floor. It would then be possible to leave an appropriate thickness of solid coal above the sheared zone as development work proceeds.

10. CONCLUSIONS

The exploration and geological interpretation done to date on the proposed development area provides an accurate and reliable representation of the geology of that area, and no further exploration of that area at the surface appears necessary. Further work in the underground development would be of major benefit to avoid exposing the sheared zone at the floor of No. 4 Seam.

Three seams, No. 11, 10 and 4, are targets for underground mining but the variation of the thickness of No. 11 Seam suggests that No. 10 and No. 4 Seams are more attractive. No. 4 Seam appears to be the most attractive because its thickness allows greater flexibility in the event of small-scale faults and the very heavily sheared top of No. 10 Seam will undoubtedly pose problems, at least for development drivage.

The condition of No. 4 Seam as exposed in the existing underground development indicates that this seam should not pose problems for longwall mining in this area, as long as an acceptable thickness of solid coal is maintained as a floor on top of the sheared coal layer.



LOG CHARACTERISTIC LINES

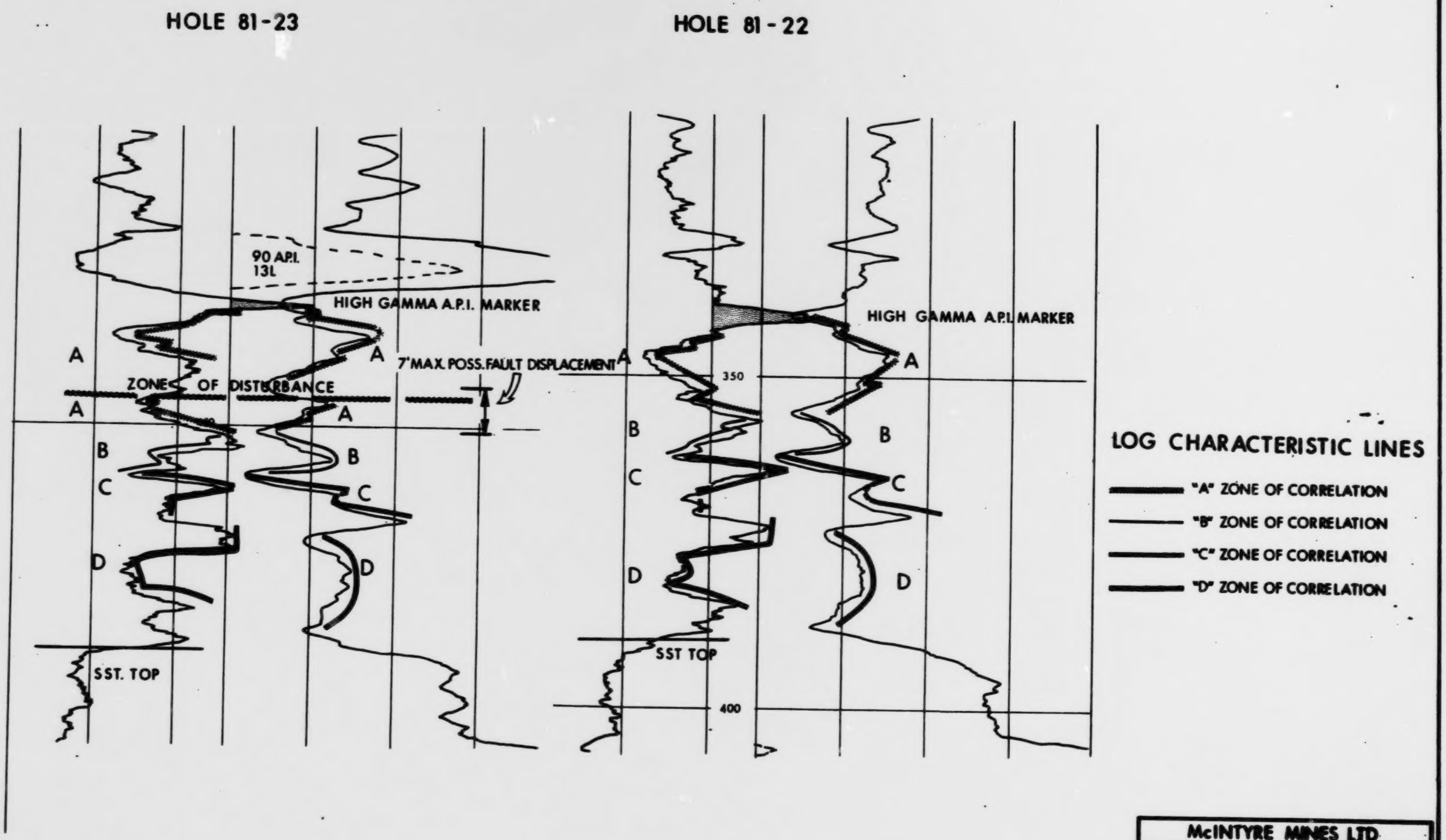
━━ "A" ZONE OF CORRELATION
── "B" ZONE OF CORRELATION
━━ "C" ZONE OF CORRELATION
━━ "D" ZONE OF CORRELATION
▬▬ "E" ZONE OF CORRELATION

McINTYRE MINES LTD.
LONGWALL MINING STUDY

FAULT ANALYSIS

JML	GRJ	SEPT. '82
HGS		
Norwest		822352040
		PLAN Nº 1

1



McINTYRE MINES LTD

LONGWALL MINING STUDY

FAULT ANALYSIS

GRJ

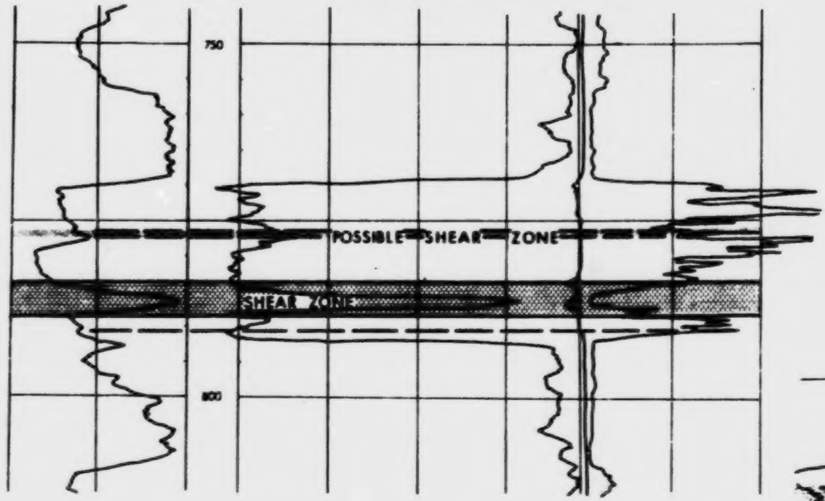
HGS

SEPT '82

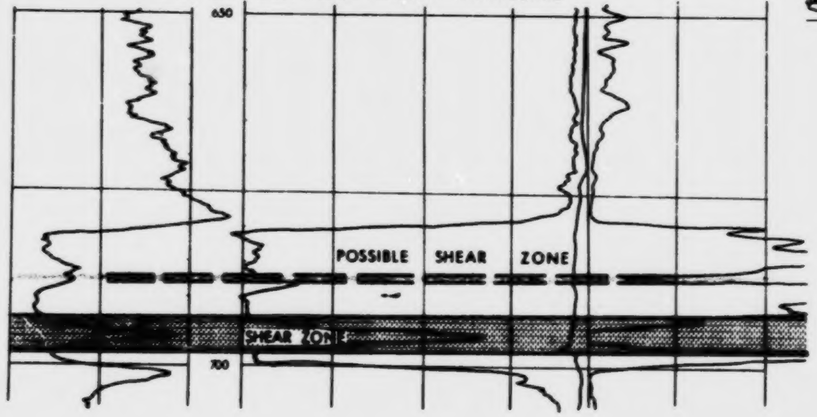
PLAN No 2

LOW SHEARING

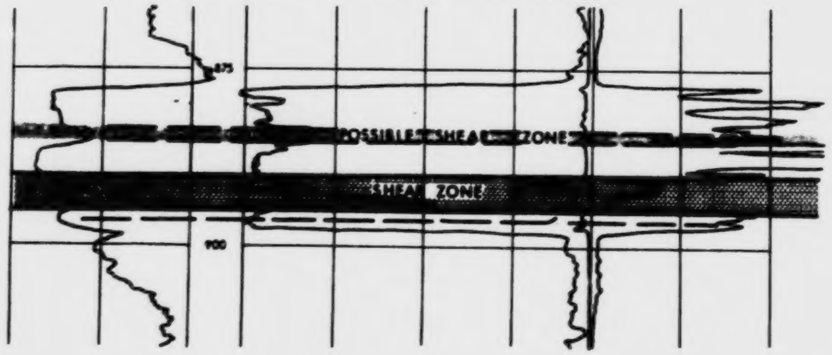
80-08 CLOSEST TO WORKINGS AT THE END OF 9G



76-149 EAST LIMB BARRET ANTICLINE

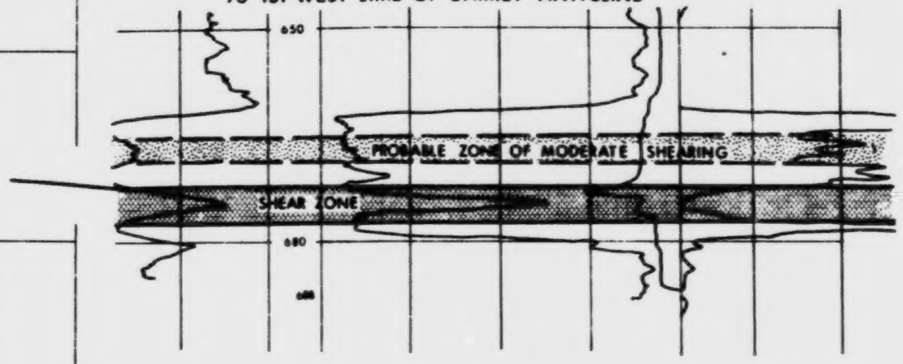


80-07 EAST DIPPING LIMB OF TAYLOR SYNCLINE



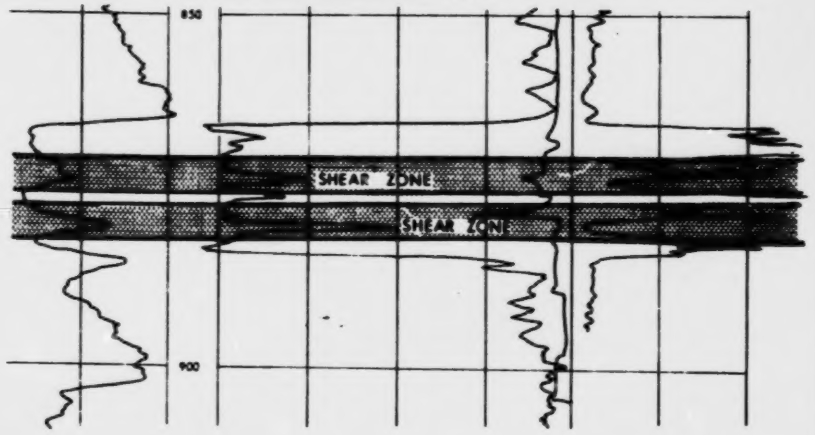
MODERATE SHEARING

76-131 WEST LIMB OF BARRET ANTICLINE



INTENSE SHEARING

80-06 AXIS OF BARRET ANTICLINE



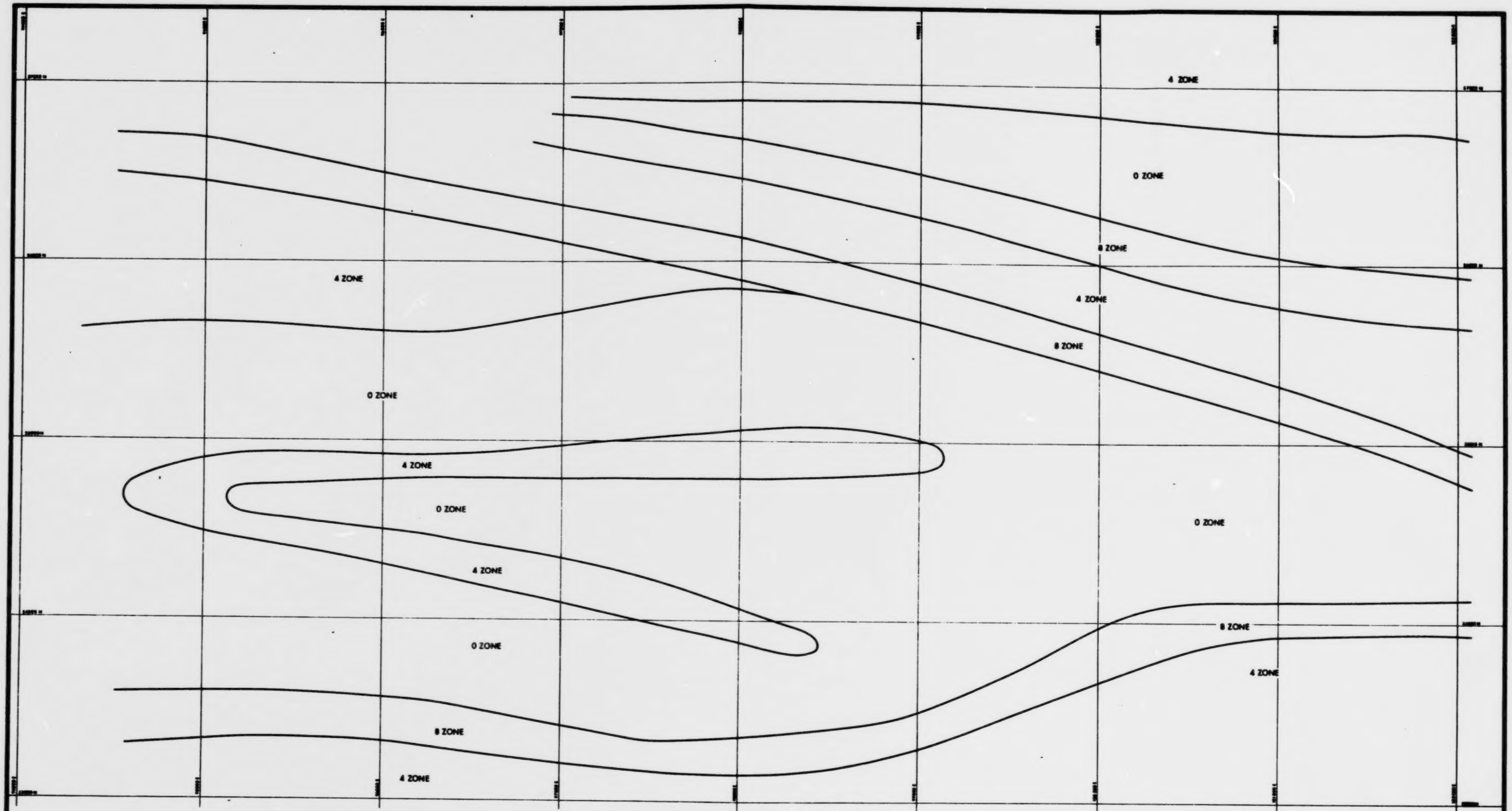
NOTE: CAVING ON CALIPER LOG & REDUCTION OF RESTIVITY
ON FOCUSED RESTIVITY LOG INDICATE SHEAR ZONE PRESENCE.

LEGEND

- ■ SHEAR ZONE
- ▨ PROBABLE SHEAR ZONE
- ▤ POSSIBLE SHEAR ZONE

McINTYRE MINES LTD.
LONGWALL MINING STUDY
SHEAR ZONE ANALYSIS
SEAM Nº 4

JML | GRJ | SEPT. '82
HGS
822235205
PLAN Nº 3



LEGEND

8 - FOLD AXIAL REGION
4 - STEEP PITCH OR GENTLE FOLD FLEXURE
0 - GENTLE DIPPING STRATA

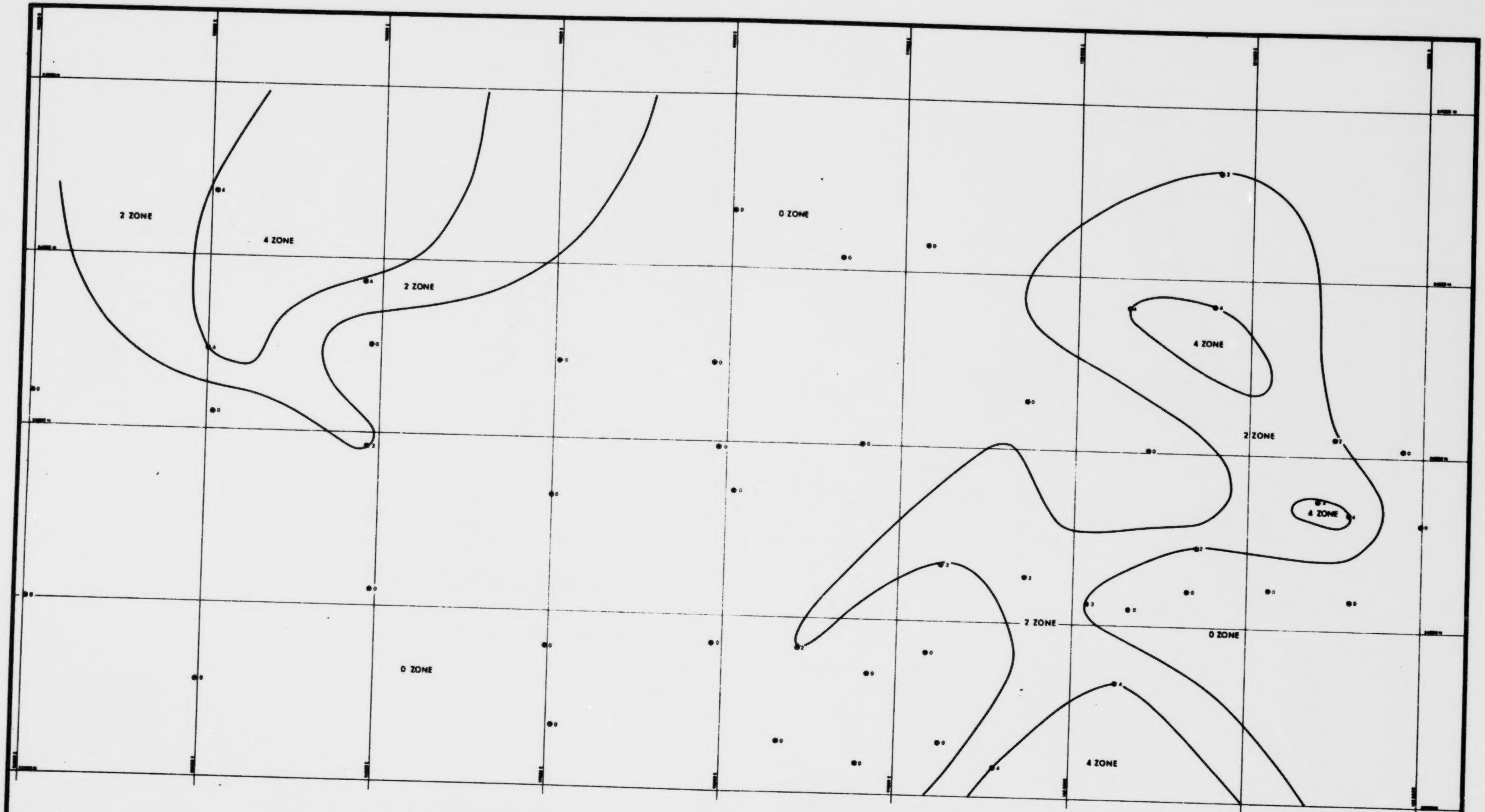
SCALE

McINTYRE MINES LTD.

LONGWALL MINING STUDY

CHANGE OF BEDDING AND PITCH No. 4 SEAM

M.J.
H.G.S.
G.E.J.
SEPT. [illegible]
1:[illegible]
Norwest
PLAN Nº 4



NOTE: THE DATA IS BASED ON DRILLERS AND LOGGING ENGINEERS REMARKS.

LEGEND

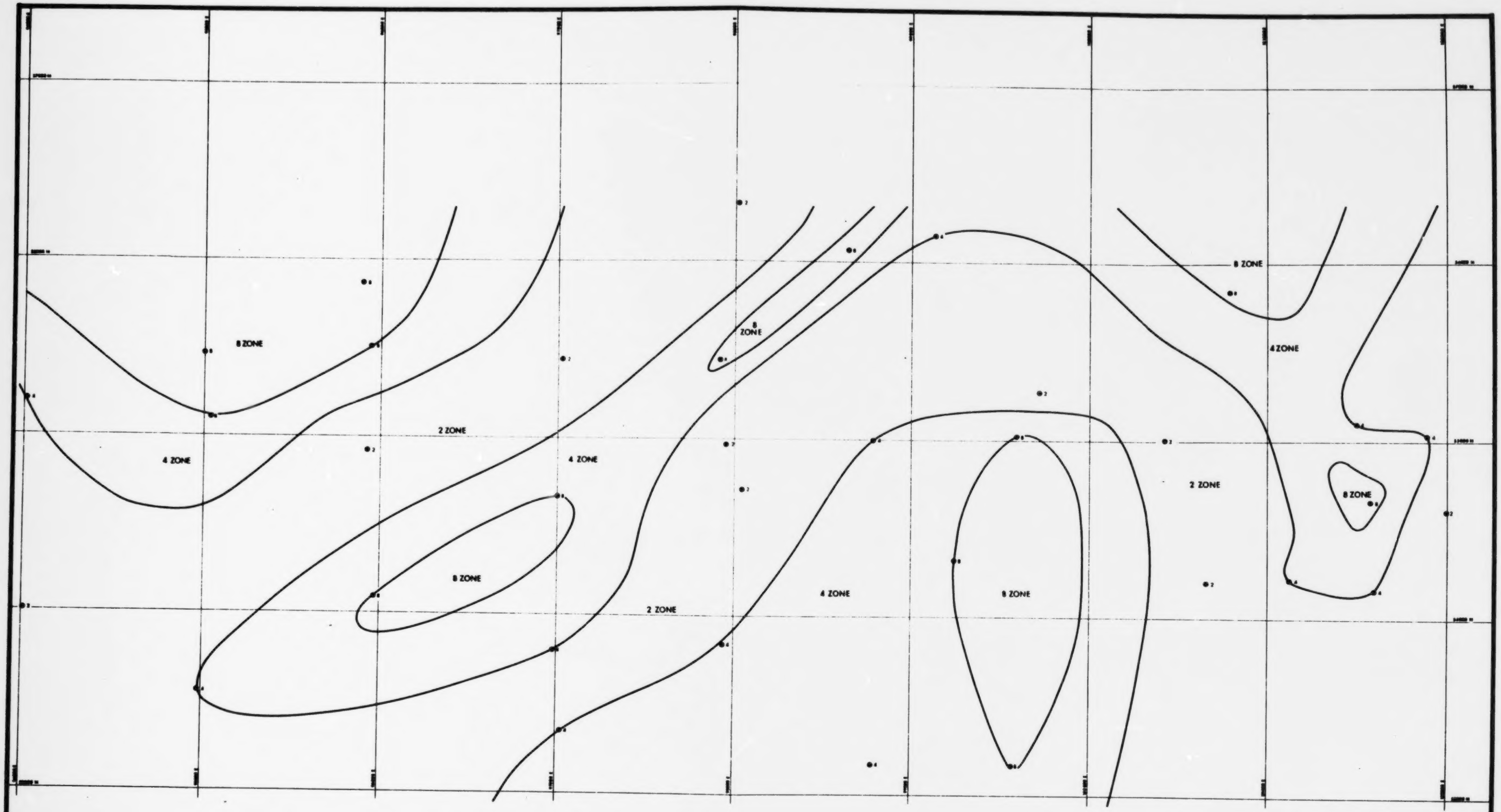
DRILL HOLE CONDITION
0 GOOD
2 FAIR
4 BAD

McINTYRE MINES LTD.

LONGWALL MINING STUDY

DRILL HOLE CONDITION
No. 4 SEAM

M.J.
G.R.J.
H.G.S.
SEPT.
PLAN Nº 5



LEGEND

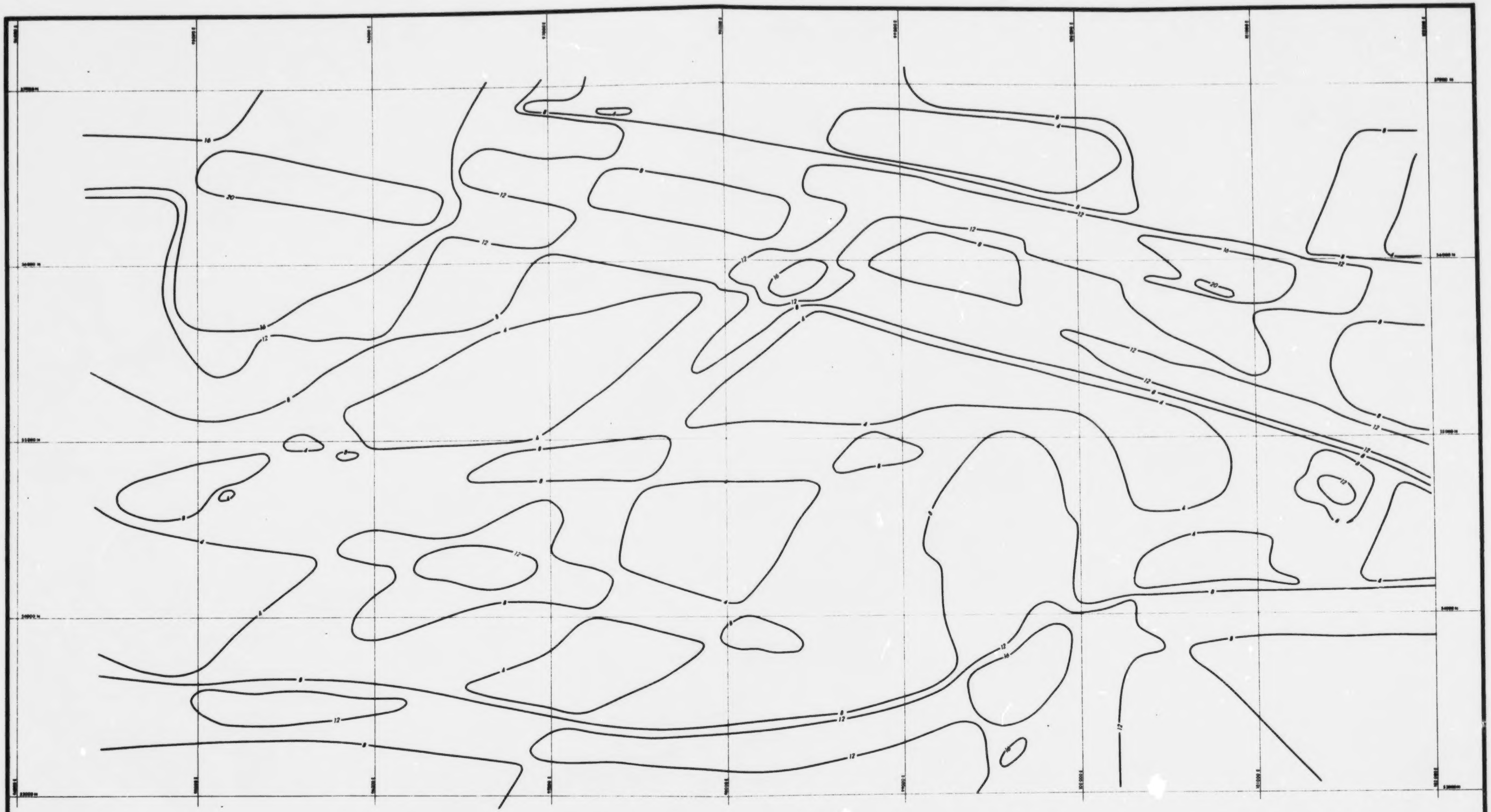
1 - UNCAVED
2 - LIGHTLY CAVED
4 - HEAVILY CAVED
8 - VERY HEAVILY CAVED

McINTYRE MINES LTD.

LONGWALL MINING STUDY

CALIPER LOG CAVING
No. 4 SEAM

M.J.
H.G.S.
G.R.J.
SEPT. '[illegible]
1:2[illegible]
PLAN Nº 6



SCALE

NOTE: CONTOUR VALUES HAVE NO NUMERICAL SIGNIFICANCE. THEY SIMPLY SHOW DIRECTION OF INCREASING GEOLOGICAL HAZARD

McINTYRE MINES LTD.

LONGWALL MINING STUDY

HAZARD ANALYSIS PLAN
No. 4 SEAM

M. J.
G. R. J.
SEPT. '82
H.G.S.
1: 200
8223S2020
PLAN Nº 7

APPENDIX 3-2

AN APPRAISAL OF THE ROOF STRATA OF THE NO. 4 SEAM IN THE AREA PLANNED FOR LONGWALL DEVELOPMENT

NORWEST RESOURCE CONSULTANTS

AN APPRAISAL OF THE ROOF STRATA OF THE NO. 4 SEAM IN THE AREA PLANNED FOR LONGWALL DEVELOPMENT

THE ASSOCIATION OF
PROFESSIONAL ENGINEERS,
GEOLOGISTS and GEOPHYSICISTS
OF ALBERTA
PERMIT NUMBER
P 914
NORWEST RESOURCE
CONSULTANTS LTD.

Prepared For: McIntyre Mines
Prepared By: Norwest Resource Consultants Ltd.

Report No.: 82-235/3

October, 1982

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PLAN NO. 1 CORRELATION CHART, ROOF STRATA NO. 4 SEAM

1. INTRODUCTION

The following report has been prepared at the request of McIntyre Mines Ltd. by Norwest Resource Consultants Ltd. The report addresses the immediate roof strata of the No. 4 Seam with particular emphasis on the presence or absence of massive sandstone which could have an impact on mine planning.

This report is based on data derived from borehole geophysical logs from a number of drillholes which are considered to be representative of the roof strata throughout the area planned for development.

Numerous other drillholes with borehole geophysical logs suitable for the investigation of this geological problem are available within this area, but these data have not been included in this appraisal. That information would be necessary for a definitive study of the roof strata, but the purpose here is simply to clarify the geological nomenclature that has been used by different geologists to describe the roof material, so that the appropriate engineering decision can be made. Because it appears that misunderstanding of terminology has arisen, a particularly lengthy discussion is included in this report so that a clear explanation can be provided. Normally it is not necessary for geologists, when conversing with each other, to discuss these aspects at this length; in a multi-disciplinary situation such as this one where this particular problem is so significant to the successful implementation of the project, it is clear that the additional discussion of the background geological data is desirable.

2. METHODOLOGY

The appraisal has been made using borehole geophysical logs from seven drillholes. Six of these holes are of the open-hole rotary drilling type, while the seventh is a diamond core drillhole. The core description data are also included in this report. The borehole geophysical logs of an eighth hole, drillhole 81-29 were also supplied but it has been shown that this hole penetrated only to No. 10 Seam and is not representative of the segment of strata immediately overlying No. 4 Seam.

The most useful geophysical logs to address this geological problem are the gamma-ray neutron, and caliper logs that have been provided for six of the drillholes. Nos. 81-12, 81-14, 81-15, 81-16, 81-20 and 81-24, inclusive. Hole No. 76-130 is also included in this appraisal, but only gamma-ray, density and caliper logs were available for that drillhole. It should be noted that the geophysical log suite used to appraise this geological problem differs from that used to appraise the zone of shearing within No. 4 Seam as documented in the report recently submitted to McIntyre Mines Ltd. This fact emphasizes the need to produce full suites of geophysical logs during early exploration phases so the multitude of geological problems that may be encountered can be accurately addressed.

The procedure followed in this appraisal was firstly to correlate the geophysical logs of the holes listed above for the interval of strata between the base of the No. 10 Seam and the top of the No. 4 Seam. The major lithological units were correlated to ensure that a more detailed correlation of immediate roof of No. 4 Seam maintains those units in the correct relationship with respect to each other, and to determine the overall degree to which sedimentation changed laterally.

A correlation diagram of the immediate roof of No. 4 Seam was then prepared (Figure 1). This is a detailed correlation but it will be noted that there are few lines shown joining the various rock units from one hole to the next. It is possible to prepare a similar diagram with many lines of correlation drawn through an equivalent stratigraphic interval. However, the geophysical logs indicate the changes that occur are often of a gradational type and, although one lithological unit may change from mudstone to siltstone and finally to sandstone for example, the correlation should be based on the complete depositional units regardless of the internal variation of those units. The details of the variation within each unit were determined from the variations shown by the gamma-ray and neutron log traces. The significance of the preceeding discussion to longwall mining is discussed in detail in Section 4 of this report.

3. THEORETICAL CONSIDERATIONS

3.1 THE INTERPRETATION OF GAMMA-RAY AND NEUTRON LOGS

The following short discussion on the interpretation of gamma-ray and neutron logs is provided so that the mining implications of the log traces, and indeed the logging methods themselves, can be understood by engineers who are not familiar with this exploration technique and the information it provides.

Gamma-ray logs are no more than a recording of the natural gamma-radiation emitted from materials within the borehole and the strata surrounding the borehole. Potassium uranium, and thorium produce natural gamma-radiation. In coalfield strata, only potassium is significant. Potassium is contained in clay minerals, so the greater the clay content of any lithological unit, the greater are the number of ratioactive decay events that are recorded by a gamma-ray detector. In coalfield strata, clays are more abundant in lithologies such as claystone, mudstone, etc., and become decreasingly common in siltstones and then sandstones. Gamma-ray logs are normally set up so that increasing decay events are recorded towards the right; fewer events are recorded towards the left. The measure of these events is called the API Unit, and the scale at the top of the gamma-ray log indicates the direction in which the API Units increase.

It will be seen from the above discussion that a coal seam that is clean and contains very little clay should produce low numbers of radioactive decays and therefore the should produce a response on the log which lies towards the left-hand side of the log trace. This can be seen with a response on gamma-ray logs on Nos. 11, 10 and 4 Seam.

As a result, the gamma-ray log is often referred to as the lithogical log because by inference it usually differentiates between sandstone with a low count value and shales or mudstones with a high count value; normally siltstones fall somewhere in between. If there is an abrupt change from one rock type to the next, there is an equivalent abrupt change shown on the gamma-ray log; if there is a gradational change from one rock type to the next, an equivalent gradational response is shown on the gamma-ray log.

Often people who are unfamiliar with geophysical logs interpret the small "squiggles" on the log trace as being indications of different beds. This is not the case, because those small "squiggles" do not relate to the nature of the lithologies adjacent to the drillhole wall; the small "squiggles" are a product of the random nature in which radioactive decay occurs. If a long enough time for sampling at any one point in a drillhole is provided, then the impact of these "squiggles" is reduced. This effect is referred to as statistical variation, and on the logs provided by McIntyre Mines the statistical variation is about a quarter of a log division.

Neutron logs act in an entirely different way than gamma-ray logs do, and they respond to the strata in a different manner. The neutron logging tool includes a source which essentially provides a supply of neutrons. The neutrons are fired into the strata and through collisions with the surrounding atoms are counted at a detector in the tool when they bounce back to that detector. The larger the mass of the atom with which they collide the greater is the chance that a neutron will be counted at the detector. For small atoms however, much of the energy of the neutron is transfered into providing energy of motion for that atom, so the neutron loses an equivalent amount of motion and therefore has a greater chance to loose speed and not return to the detector. In coalfield strata, hydrogen is of course by far the smallest of the atoms present, and the size of the neutron is approximately the same as that of hydrogen, and so in a collision between hydrogen and a neutron there is a very high probability that the neutron will not maintain sufficient energy to return to the detector to be counted. This means that where there is a high concentration of hydrogen present in the strata, neutrons have a low chance of returning to the detector to be counted, and consequently low neutron values are recorded for hydrogen-rich lithologies.

Any hydrocarbon including coal has a high concentration of hydrogen, so these materials produce a low neutron response. This can be seen from the response to coal in Seams Nos. 4, 10 and 11 on the neutron logs of the drillholes provided by McIntrye. Clay minerals also tend to have a high concentration of bound hydrogen atoms and therefore claystones and mudstones tend to give a low neutron response. Naturally, rock types with a increasing amount of water tend to give progressively lower neutron values, so a sandstone with a high porosity will produce a lower neutron response than one which has a low porosity. A very clean, well-cemented massive sandstone can therefore be expected to produce high neutron counts. This kind of response can be seen for some of the sandstone units contained about 50 feet above No. 4 Seam. The preceeding discussion of the neutron tool is not theoretically precise but it does provide a fundamental understanding of the meaning of the responses on this log.

3.2 THE TERM "MASSIVE"

The use of this terms has entirely different meanings for geologists and for engineers, particularly those involved in longwall mining. For a geologists the term is applied to any lithology which does not appear to be bedded. The term is therefore not one which is restricted to sandstone but should, for example, normally be applied to many of the marine mudstones that are observed in the Lower Cretaceous strata of Western Canada. From a geological point of view, it is also not restricted by the size of the bed, but is applied to the internal description of any lithological unit of any selected thickness. For longwall mining engineering the term "massive" is often applied to all thick sandstone units which may or may not be bedded. These sandstone units are ones which have the particularly unfortunate characteristic of refusing to fail in a regular manner, causing stress to build up at the mining face. The usual characteristics of the sandstone beds causing this problem are:

(a) that they are relatively thick units;
(b) that they are well-cemented;
(c) that they have a tendency towards being non-bedded;
(d) that they consist of very competent grains, such as silica;
(e) that they have an absence of matrix or contain a very competent matrix lithology such as silica;
(f) that they are infrequently jointed;
(g) that they are infrequently fractured;
(h) that they are not interbedded with other less competent lithologies on a regular basis; and
(i) that they are lithological units with a high compressive and tensile strength.

Theoretically therefore, other lithological units beside sandstone could produce the same response for mining as do massive sandstones. This would occur for example, in a mudstone if all of the above parameters were satisfied and equivalent to a massive sandstone unit. Of course this does not occur in practice because the mudstones consist mostly of clay minerals which do not have the same degree of competency as that of the minerals normally included in sandstone. For mudstones, many of the other factors are also quite different under normal circumstances, such as the degree of jointing and layering, the porosity, and permeability.

It is obvious from the preceeding discussion that it is easily possible for misunderstandings to occur over the use of the term "massive" between geologists and mining engineers. It has been the writer's experience that less confusion arises if geology is documented without reference to this term and for a specific document to be prepared which is aimed at defining all of those units within the sequence of strata above a seam to be mined, which are predicted to behave in a "massive" manner according to an engineering definition.

4. RESULTS

The sequence of strata immediately overlying the No. 4 Seam and extending from a thickness of about 45 feet in Hole No. 81-15 to a thickness of about 75 feet in Hole 81-24 should be considered as a single depositional unit. This unit includes three sequences: the cap-rock mudstone on top of the seam at the base of the sequence, proceeding upwards through a gradational sequence of interbedded muds, silts and fine-grained sands, to a sequence of sandstone at the top which is expected to display many of the characteristics of a massive sandstone according to an engineering definition.

The log characteristics of this depositional unit, especially as displayed in Holes No. 81-20, 81-16 and 81-12, indicate that it is either a barrier-bar deposit or a deltaic sequence. In both of these models, the methods of deposition are very similar and the same conclusions regarding the nature of the deposit can be drawn. These models suggest the following:

(a) that the gradational sequence below the sandstone is well-bedded and initially nearly horizontal;

(b) that the sandstone sequence at the top of this unit is particularly well-sorted and relatively poorly bedded; and

(c) that the sequence of beds between the top of the coal seam and the base of the sandstone should be well-bedded, and therefore should not behave in a mining situation as if they were massive units.

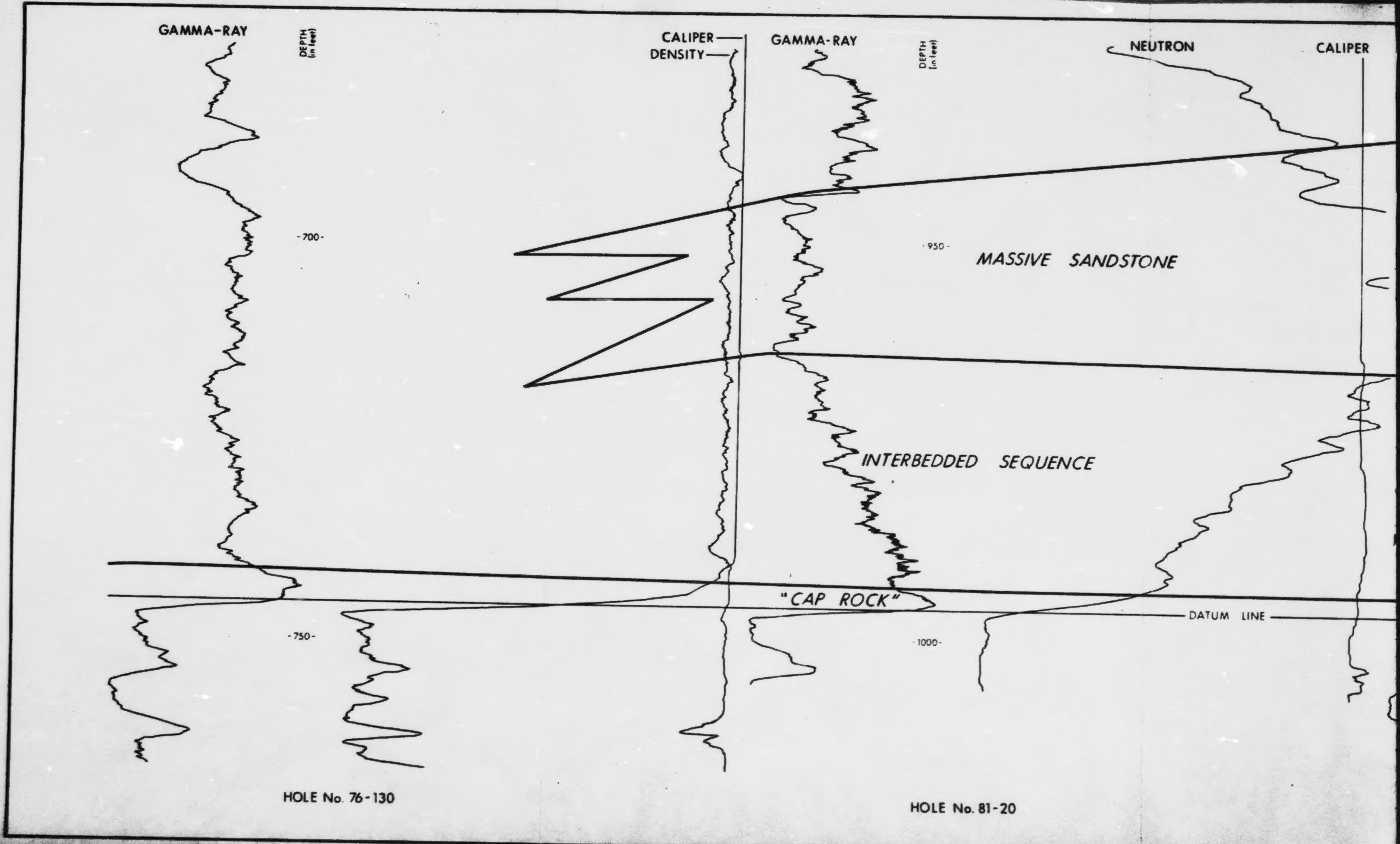
The geophysical logs indicate that the cap rock or roof mudstone of the No. 4 Seam has a gradational upper contact with the overlying interbedded silt-sand - mud sequence. Nevertheless, the approximate thickness of this unit varies from about 2 feet in Hole 81-16 to about 6 feet in Hole 81-24. The interbedded sequence and the cap-rock lying between the top of the No. 4 Seam and the base of the overlying sandstone has a minimum thickness of 19 feet in Hole 81-14 and a maximum thickness of 43 feet in Hole No. 81-24. The sandstone unit at the top of the sequence wedges out between Holes 81-20 and 76-130, but there is a sequence of about 50 feet of interbedded siltstone and mudstone overlying the No. 4 Seam in Hole 76-130. The neutron log shows high neutron counts in most cases through the sandstone unit at the top of the sequence, indicating that the sandstone unit has a low porosity; in this kind of depositional environment, a low porosity suggests that the sandstone is well-cemented. Although the sand units within the interbedded sequence do not usually show an equivalent low porosity, two sandstone beds with low porosity occur in the interbedded sequence in Hole 81-15; the thickest of these units is about 2.5 feet located approximately 15 feet above the top of the No. 4 Seam. The logs also show that the bedding contacts adjacent to these sandstone unit are abrupt, and this especially true for the upper contact of each of these sandstone phases.

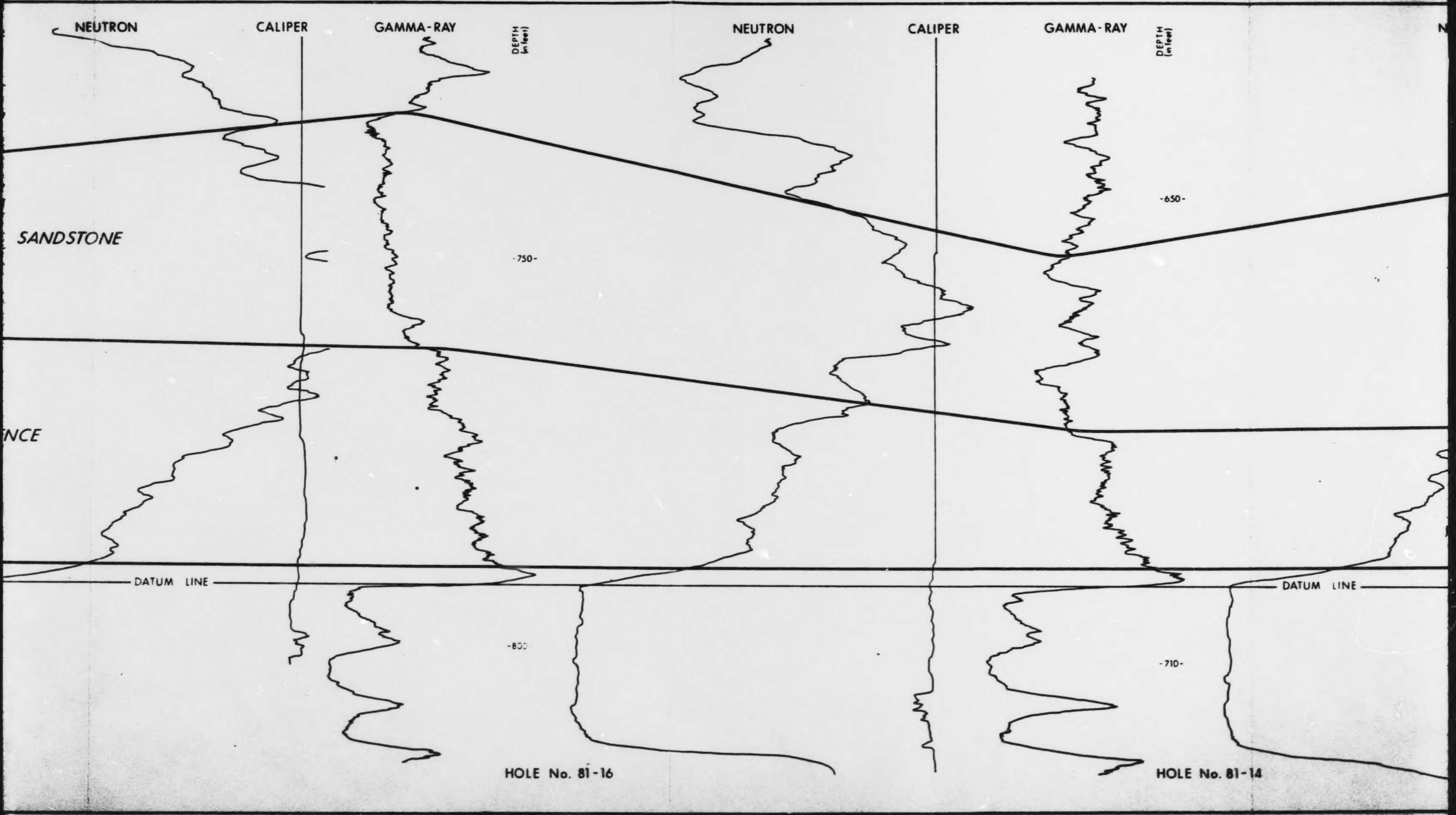
The core from Hole 81-24 provided an opportunity to investigate the nature of bedding of the immediate roof of the No. 4 Seam, but unfortunately details of this feature are not well documented on the written log. However, a dipmeter geophysical log is available, and this log shows that the sequence of strata between the top of the No. 4 Seam and the base of the sandstone unit at the top of the roof sequence is quite strongly bedded in a regular manner. In fact, that sequence of strata is shown to be probably the most clearly bedded unit in the whole of the drillhole. On this log, the interbedded sequence has the best level of correlation for dip determination for the whole of the drillhole, and in general there are only minor changes of the strike direction and relatively minor fluctuations of the angle of dip. In contrast, the dipmeter readings for the overlying sandstone show a lower level of correlation, a wide variation of strike direction and greater variation of the angle of dip; in fact there is an absence of dip measurements towards the base of that unit.

5. CONCLUSIONS

On the basis of the information supplied, Norwest believes that the roof of the No. 4 Seam consists of strata grading from mudstone at the base, through a laminated sequence of interbedded muds and silts, to an overlying relatively thick, well-cemented sandstone. The interbedded sequence and the immeadiate mudstone cap-rock varies in thickness from 19.5 feet minimum to 50 feet maximum and is strongly bedded. The sandstone unit at the top of the sequence is expected to behave as if it were massive from an engineering point of view.

Sandstone beds may occur within the interbedded sequence, but only on a localized and infrequent basis. The maximum thickness of these beds is not expected to exceed 3 feet, and they should normally have clearly defined bedding contacts, especially at the top. It is expected that the thickness of these beds will be thin enough to allow regular caving to take place, so it is anticipated that there is a minimum thickness of 19 feet of well-bedded roof material that will cave in a regular manner during longwall mining. It is predicted however, that the sandstone overlying this interbedded sequence will have the behavior characteristics of a massive unit from an engineering standpoint.


GAMMA-RAY
DEPTH (in feet)
CALIPER
DENSITY
GAMMA-RAY
DEPTH (in feet)
NEUTRON
CALIPER
-700-
-950-
MASSIVE SANDSTONE
INTERBEDDED SEQUENCE
"CAP ROCK"
DATUM LINE
-750-
-1000-
HOLE No. 76-130
HOLE No. 81-20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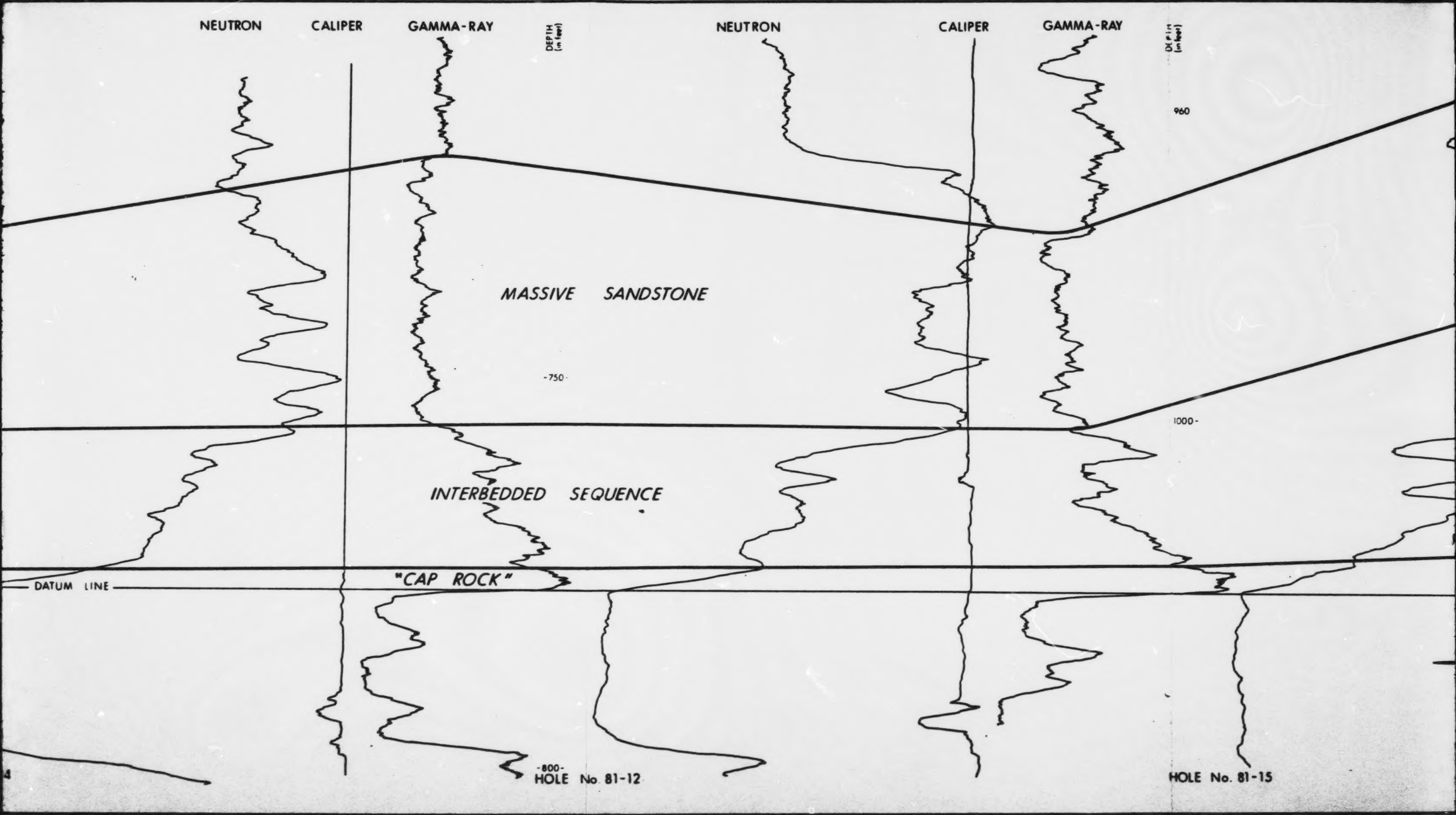
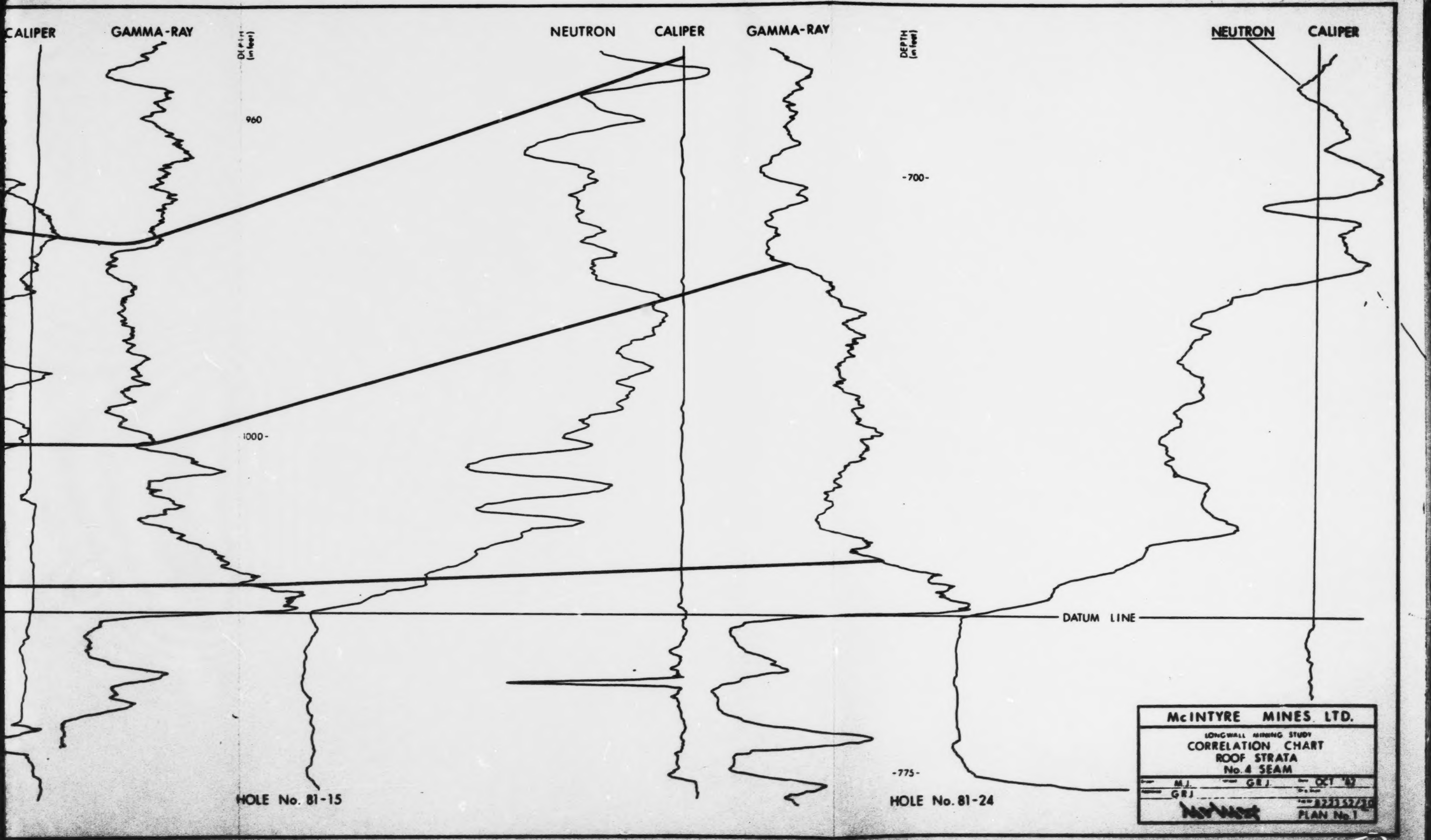
CALIPER
GAMMA-RAY
DEPTH (in feet)
NEUTRON
CALIPER
GAMMA-RAY
DEPTH (in feet)
N
-650-
SANDSTONE
-750-
NCE
DATUM LINE
DATUM LINE
-800-
-710-
HOLE No. 81-16
HOLE No. 81-14


NEUTRON
CALIPER
GAMMA-RAY
DEPTH (in feet)
NEUTRON
CALIPER
GAMMA-RAY
DEPTH (in feet)
960
MASSIVE SANDSTONE
-750-
1000-
INTERBEDDED SEQUENCE
"CAP ROCK"
DATUM LINE
-800-
HOLE No. 81-12
HOLE No. 81-15



McINTYRE MINES LTD.

LONGWALL MINING STUDY
CORRELATION CHART
ROOF STRATA
No. 4 SEAM

M.J. GRJ. OCT '82
GRJ
822352/20
PLAN No. 1

APPENDIX 3-3

GEOTECHNICAL EVALUATION LOGGING/TESTING/ANALYSIS
PROPOSED LONGWALL AREA - NO. 9G-4 MINE
GRANDE CACHE ALBERTA
SEEGMILLER INTERNATIONAL

SEEGMILLER INTERNATIONAL
MINING GEOTECHNICAL CONSULTANTS/ENGINEERS
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GEOTECHNICAL EVALUATION
LOGGING/TESTING/ANALYSIS
PROPOSED LONGWALL AREA - No. 9G-4 MINE
GRANDE CACHE, ALBERTA

PREPARED FOR

McINTYRE MINES LTD.
Calgary, Alberta

By

Dr. Ben L. Seegmiller
Principal Consultant

November 1982

Copy 8 of 13

Approved: Ben L Seegmiller 11-11-82
Ben L. Seegmiller, P.E.#3485 Utah, P. Eng. #34141 Alberta
President, Seegmiller International

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SUMMARY

In brief, this study has involved several site visits, geotechnical logging and testing of drill cores and reviews of available data applicable to the proposed longwall panels of the No. 9G-4 Mine at Grande Cache. The following conclusions have resulted:

1 - The floor bearing capacity may be acceptable provided the soft, shear coal or blackjack does not occur as the immediate floor.

2 - The support load density for a typical longwall panel is predicted to be on the order of 7.5 - 8.0 Tsf with a corresponding bearing pressure of 300 psi or less depending on the equipment model.

3 - Ongoing engineering and research projects should include (a) chain pillar design, (b) drilling and strength testing, (c) subsidence considerations and (d) displacement/stress monitoring.

INTRODUCTION

A geotechnical evaluation of certain aspects of the proposed longwall area of the No. 9G-4 Mine, Grande Cache, Alberta is the subject of this report. Mr. Howard D. Owen, Manager of Engineering, McIntyre Mines Limited, Calgary, Alberta authorized the study and coordinated the site visits and core shipments.

McIntyre Mines Limited(hereinafter referred to as McINTYRE) is in the process of investigating the possibility of a demonstration longwall project at their No. 9G-4 Mine. The first phase of the project involves a number of considerations of which a geotechnical evaluation is part. Toward that end, Seegmiller International(hereinafter referred to as SEEGMILLER), was contracted to provide geotechnical expertise in the initial phases of the project. The present geotechnical study was begun in latter March 1982 with a site visit. Field data were collected during a June 1982 SEEGMILLER site visit and drill cores were shipped to Salt Lake City for strength testing in September 1982. Original plans had called for roof, coal and floor coring in the No. 9G-4 Mine, but subsequent drill availability problems ruled out such an undertaking. The present analyses and report are the culmination of the SEEGMILLER geotechnical evaluation.

The basic purposes of the study and of this report are to (1) provide an assessment of the bearing capacity of the materials in and around the No. 4 Seam, (2) evaluate the roof caving height and caving characteristics and (3) provide a preliminary evaluation of the support load densities for the longwall equipment. The study and report are limited to (1) an evaluation of rock cores from four boreholes located in the proposed panel areas, (2) on-site observations in the No. 9A and No. 9G-4 Mines and (3) a review of previous laboratory and in-situ test reports. Additional information was provided by McINTYRE and Norwest Resource Consultants Ltd. personnel.

This report begins with an evaluation of the material properties including the geotechnical core logging and laboratory testing and a review of previous strength testing projects. Longwall geotechnical evaluations are then undertaken. These evaluations include bearing capacity, caving and support load densities. Some engineering and research recommendations are then presented and pertinent conclusions complete the report. Detailed geotechnical logs and laboratory test results are appended.

MATERIAL PROPERTIES

GEOTECHNICAL CORE LOGGING

General. Geotechnical logs were prepared for roof and floor core materials from four representative boreholes, including drill holes C81-22, C81-23, C81-24 and C81-29. The roof zone logged began between 37.0 and 61.5 feet above the No. 4 Seam and continued down to the coal seam. The No. 4 Seam materials had been removed for analysis and were thus, not available. The floor zone began at the base of the No. 4 Seam and continued downward between 20.0 and 25.5 feet. All of the logging was done on site at Grande Cache.

Properties Logged. A total of 282.5 feet of core, including 196 feet of roof and 86.5 feet of floor, was logged for geotechnical properties. Items logged include the following: recovery(R), rock quality designation(RQD), fractures, fracture intersection angle(α), fault zones, hardness, weak and strong zones and partings. Both graphic and descriptive logs were prepared. The graphic logs show the rock types, discontinuities, broken zones and the location of the samples used in strength testing. The descriptive logs are divided into zones of similar materials with similar geotechnical characteristics.

Logged Results. The individual graphic and descriptive logs are presented in Appendix I. Included as a preamble to the logs are explanations of some of the technical codings and designations used in logging. Details of each roof and floor unit are given, together with a summary of the expected conditions at that point in the roof and floor.

Results Discussion: ROOF. The lowermost roof of the No. 4 Seam is a dense, hard material locally referred to as caprock. It is predominantly a carbonaceous siltstone and occasionally a claystone. The caprock varies in thickness from ½ to 1 inch and is overlain by thin coal riders which are generally about 1 inch thick. These materials have proven unsupportable in the past and, therefore, up to 1 foot or more of the roof has been removed during coal mining. Overlying claystones are strongly slickensided, but appear to be very strong and stand well with rock bolt support. Protection from thin slabbing of the roof materials has been accomplished using wire mesh in conjunction with the roof bolts. The roof, 10 to 15 feet above the No. 4 Seam, increases in strength very significantly. At a distance of about 30 feet above the No. 4 Seam, the rock is 3 to 10 times stronger than that forming the lowermost five feet of roof.

Results Discussion: FLOOR. The immediate floor materials are quite variable in strength ranging from fairly weak claystones to very strong sandstones and siltstones. The stronger materials do seem to predominate, at least in three of the four drill holes examined. Wet conditions could be adverse if the weak claystones widely occur, but if the stronger sandstones and siltstones predominate, no problems are expected.

LABORATORY STRENGTH TESTING

General. Laboratory tests were used to estimate various strength parameters of available roof, coal and floor materials. The tests were performed using standard laboratory sample preparations and testing procedures. A description of the types of tests performed is presented in the following paragraphs.

Uniaxial Compressive Strength. The uniaxial compressive strength, which is one of the most common physical properties usually cited, is determined by placing rock cores having length to diameter ratios of approximately 2 to 3.5 in a standard testing machine and loading them to failure. The uniaxial compressive strength (U) is computed as follows:

$$U = \frac{P}{A} \qquad \{1\}$$

where P = load at failure
A = upper surface area

Elastic Modulus. The tangent modulus was determined using the axial displacement recorded in the rock during the uniaxial compressive test. The calculation for the tangent modulus is performed using the following formula:

$$E_t = \left(\frac{\Delta\sigma_z}{\Delta\varepsilon_z}\right)_{\sigma_1} \qquad \{2\}$$

where $\Delta\sigma_z$ = change in stress
$\Delta\varepsilon_z$ = change in strain
σ_1 = stress, usually at 50% failure stress

Direct Shear Strength. The direct shear strength of a rock core may be envisioned as the shear strength which results when the normal stress is zero. In other words, the shear strength of a rock could be approximated as the cohesion at zero normal stress. Therefore, an estimate of cohesion may be made by testing a rock core in direct shear. The specific test used in the study is known as the double shear test and is defined as follows:

$$D = \frac{P}{2A} \qquad \{3\}$$

where P = load at failure
A = cross sectional core area

LABORATORY STRENGTH RESULTS

Test Sample Sources. The sample sources for the roof and floor materials included core from drill holes C81-22, C81-23 and C81-24. Additional PLT tests were conducted on miscellaneous lump roof samples and lump roof caprock. The coal uniaxial compressive and modulus tests were performed on samples cored from blocks of hard coal obtained from the

No. 9G-4 Mine. The coal PLT tests were performed on irregular lumps left over from coring.

Test Results. Computed statistics for the test results are presented in Table I. Similar statistics for various rock types are presented in Table II. A summary of the PLT test results is shown in Table III. Detailed sample descriptions and test results are presented in Appendix II.

Results Discussion: ROCK. The roof and floor rocks are rated, in general, as moderately strong to strong. The rocks in the roof are probably stronger than those in the floor, particularly those rocks above the immediate roof. Sandstones are the strongest rock type followed by siltstones and claystones. Actual in-situ rock strength may be higher because the samples tested had been stored in unsealed containers for more than a year.

Results Discussion: COAL. Basically, the only coal tests that could be performed were two uniaxial compressive tests during which the elastic modulus was measured. Based on these tests, the coal must be considered to be of very low strength. Further, during the coring process, vibration and water was noted to have a very drastic disintegration effect on the freshly cut coal.

PREVIOUS TESTING PROGRAM (1976)

Background. In 1976, a geotechnical program was undertaken by R. M. Hardy & Associates Ltd. of Calgary, Alberta. In part, the purpose of the program was to develop basic mechanical rock properties for analytical design and stability evaluation of proposed mine pit walls for the No. 9 Mine Extension. The basic rock properties are believed significant to the present study and, therefore, were made available by McINTYRE.

Test Results Summary. A summary of average test values for bulk density, uniaxial compressive strength, elastic modulus and Poisson's ratio is presented in Table IV. Average PLT test results for different rock types are shown in Table V.

Results Discussion. The test results are, in general, in relative agreement with those of the present study. The sandstones are strongest, followed by siltstones and mudstones.

TABLE I

PHYSICAL PROPERTY TEST RESULTS: ROOF, FLOOR & COAL

Physical Property	ROOF			FLOOR			COAL		
	N	R	$\bar{x} \pm S_x$	N	R	$\bar{x} \pm S_x$	N	R	$\bar{x} \pm S_x$
Uniaxial Compressive Strength,MPa	1	56.4-56.4	56.4±0.0	3	59.0-105	79.5±23.4	2	0.80-1.29	1.04±0.35
Elastic Modulus,GPa	0	N/A	N/A	0	N/A	N/A	2	1.11-1.62	1.36±0.36
Direct Shear Strength,MPa	2	4.31-6.71	5.51±1.70	3	10.5-20.2	17.3±5.88	0	N/A	N/A

TABLE II

PHYSICAL PROPERTY TEST RESULTS: BY ROCK TYPE

	Uniaxial Compressive Strength,MPa			Direct Shear Strength,MPa		
Rock Type	N	R	$\bar{x}\pm S_x$	N	R	$\bar{x}\pm S_x$
Sandstone	3	59.0-105	79.5±23.4	1	21.1-21.1	21.1±0.0
Siltstone	1	56.4-56.4	56.4±0.0	1	20.2-20.2	20.2±0.0
Clayey Siltstone	0	N/A	N/A	1	10.5-10.5	10.5±0.0
Carbonaceous Claystone	0	N/A	N/A	2	4.31-6.71	5.51±1.70

TABLE III

PLT TEST RESULTS: SUMMARIES

Source	I(AXIAL),MPa	I(DIAMETRAL),MPa	I(ANISOTROPY)
C81-22 Roof	5.6	5.4	1.0
C81-23 Roof	5.8	1.5	3.9
C81-24 Roof	7.5	1.7	4.4
Miscellaneous Roof	4.7	1.5	3.1
Roof Caprock	2.6	1.3	2.0
Coal	0.48	0.49	1.0
Claystone	2.3	0.7	3.3
Siltstone	6.0	2.0	3.0
Sandstone	7.5	4.0	1.9

T A B L E I V

PHYSICAL PROPERTY TEST RESULTS: 1976 STUDY

Material	Bulk Density,pcf	Uniaxial Compressive Strength,MPa	Elastic Modulus,GPa	Poisson's Ratio
Sandstone(fresh)	167.7	78.5	N/A	N/A
Sandstone(medium-grained)	N/A	N/A	55.9	0.14
Sandstone(fine-grained)	N/A	N/A	51.0;53.8	0.12;0.16
Siltstone	160.4	61.1	33.1	0.16
Mudstone	158.0	7.9	N/A	N/A
Coal(weak, wet)	83.7	0.0	N/A	N/A
Coal(strong, banded)	93.5	7.8	N/A	N/A

TABLE V

PLT TEST RESULTS: 1976 STUDY

Material	I(AXIAL),MPa	I(DIAMETRAL),MPa	I(ANISOTROPY)
Sandstone(medium-grained)	16.8	4.6	3.6
Sandstone(fine-grained)	10.7	3.6	3.0
Siltstone	9.3	1.5	6.2
Mudstone	3.1	1.5	2.1

The actual uniaxial compressive strength magnitudes are very similar for corresponding sandstones and siltstones. The PLT test results are higher for the 1976 study. This indicates that fresher, more unweathered samples were tested. The coal materials are again very weak when the lower strength, wet coal is considered.

PLATE LOADING TESTS

Background. Some plate loading tests were undertaken by the Coal Mining Research Centre (CMRC) at the No. 9G-4 Mine in June 1982. The purpose of these tests, according to CMRC Report[1] #82/46-C, was to determine the compressive strength of the mine floor of the No. 4 Seam. A total of 19 tests was conducted in some four different horizons of the No. 4 Seam. According to the CMRC report, the four horizons, beginning just below the caprock roof, are in order, hard coal(11.5 ft.), soft shear coal(5.5 ft.), medium-hard coal(2.5 ft.) and hard coal(1.5 ft.). A sandstone was said to underlie the lowermost hard coal. The tests were conducted using a 100-ton hydraulic ram, various ancillary equipment and a 12-inch by 12-inch bottom loading plate.

Test Results Summary. A summary of the 19 plate loading tests is presented in Table VI. Owing to the fact that the soft, shear coal presents the greatest concern, relative to strength, most of the tests were conducted on that coal horizon.

Results Discussion. The results appear to be on the order of what should be expected for the No. 4 Seam coals. Further, the extremely low values for the soft, shear coal (blackjack) are believed to be some of the lowest bearing capacities ever reported for coal materials. Although the harder coals have greater bearing capacity magnitudes, they are also considered relatively low for North American coals. Overnight and weekend tests were reportedly made, but the details of these tests are sketchy.

TABLE VI

PLATE LOADING TESTS: CMRC RESULTS SUMMARY

Depth, ft.	Material	Thickness	No. of Tests	Yield Strength, psi	Ultimate Strength, psi	Plate Penetration, inches
	CAPROCK					
0.0	HARD COAL	11.5'	4	600+	900+	0.23-2.69
11.5	SOFT SHEAR COAL (BLACKJACK)	5.5'	12	100-300	130-215	2.33-6.46
17.0	MEDIUM HARD COAL	2.5'	2	630	660+	0.28-4.83
19.5	HARD COAL	1.5'	1	475	600+	0.94
21.0	SANDSTONE					

LONGWALL GEOTECHNICAL EVALUATIONS

BEARING CAPACITY

Bearing Capacity Theory. A method of calculating the bearing capacity of a horizontal surface was originally formulated by Terzaghi[2] in 1943. The method is based on rigorous theory which states that the bearing capacity is a function of the loading width, the density and internal friction and cohesion of the material comprising the horizontal surface. Further, if a surcharge load is placed immediately adjacent to the bearing surface, the bearing capacity is increased. The method may be reasonably applied in soft or weak rocks having internal friction angles less than 45°. The general bearing capacity equation[3] takes the following form:

$$q_f = 0.5\gamma B H_\gamma + C N_c + q N_q \qquad \{5\}$$

where q_f = bearing capacity
γ = unit weight
B = loading width
C = cohesion
q = surcharge load
$N_q = e^{\pi \operatorname{Tan}\phi} \operatorname{Tan}^2(45 + \phi/2)$
$N_c = (N_q - 1) \operatorname{Cot}\phi$
$N_\gamma = 1.5(N_q - 1) \operatorname{Tan}\phi$
ϕ = internal friction

For mine floor applications the surcharge loads may be assumed negligible. Generally, the loading width factor is less than 1% of the cohesion factor and thus, may also be ignored. Therefore, Equation {5} may be rewritten to compute a mine floor bearing capacity(B_c) as follows:

$$B_c = C\,[e^{\pi \operatorname{Tan}\phi} \operatorname{Tan}^2(45 + \phi/2) - 1] \operatorname{Cot}\phi \qquad \{6\}$$

Bearing Capacity: Coal. From the testing conducted for this study only two uniaxial tests could be determined. These tests were on the harder coal and indicated a uniaxial compressive strength of 1.04 MPa or about 150 psi. These values are extremely low and subject to question. The PLT results indicate a $I_{(AXIAL)}$ index of 0.48 MPa or about 70 psi for the same coal. The PLT index may be related to uniaxial compressive strength by the following formula:

$$U \cong 25I \qquad \{7\}$$

Using this relationship would yield a coal strength of about 1750 psi; which may be overstating the strength of the No. 4 Seam materials. Owing to the fact that no triaxial testing could be performed, values of internal friction(ϕ) and cohesion(C) will have to be developed by other means. One method would be to assume the internal friction angle and then a cohesion could be estimated using the assumed ϕ angle and a uniaxial compressive strength. A relationship that can be derived from the Mohr-Colomb failure envelope, generally assumed to govern rock failure, is

$$U = \frac{2\ C\ \text{Cos}\phi}{1 - \text{Sin}\phi} \qquad \{8\}$$

where U = uniaxial compressive strength
C = shear strength or cohesion
ϕ = internal friction angle

Assuming that the No. 4 Seam hard coal materials have an internal friction angle of 20° would yield a cohesion of 58 psi using Equation {8} and the uniaxial compressive strength of 150 psi. The bearing capacity using Equation {6} may then be computed to be

$$B_c = (58)(6.4 - 1)\ \text{Cot}\ 20° = 860\ \text{psi}$$

This value is generally in agreement with the bearing plate test results conducted by CMRC. However, all of the factors used in these calculations may be very conservative and thus, the bearing capacity as computed by foundation theory may overestimate the in-situ bearing capacity. For the soft, shear zone or blackjack, the material properties could be as low as $\phi = 15°$ and C = 10 psi. For these values the bearing capacity would be

$$B_c = 10(3.94 - 1) \text{ Cot } 15 = 110 \text{ psi}$$

This value is in good agreement with the plate bearing tests, but again the assumed material properties are extremely low.

Roof and Floor Bearing Capacities. The weakest rock encountered during testing was the carbonaceous claystones, one of which had a direct shear strength of 4.3 MPa or approximately 625 psi. Assuming this value is representative of the cohesion magnitude and assuming an internal friction angle of 25° for these slickensided rocks, the bearing capacity would be

$$B_c = 626 \text{ Cot } 25(10.66 - 1) \approx 13{,}000 \text{ psi}$$

This value is believed to be quite high for the weak carbonaceous claystones, but even if it is high by a factor of 5, it is still indicative that few, if any, problems will be found in the roof and floor rocks.

Typical Floor Bearing Capacities: USA Mines. The United States Bureau of Mines conducted bearing capacity tests in some 9 bituminous coal mines in the USA in 1969-1970. A summary[4] of these test results is presented in Table VII. It should be noted that the Colorado mine, the Illinois mine and the West Virginia(1) mine did not have longwalls in use. The Colorado mine had a bearing capacity of only 490 psi and the West Virginia(1) mine had a bearing capacity of 1600 psi. All other mines had floor bearing capacities of at least 2000 psi.

TABLE VII

BEARING CAPACITY TEST RESULTS: USBM STUDIES

Mine Location	Cover, ft.	Seam Thickness, inches	Extraction Height, ft.	Floor Geology	Average Bearing Capacity, psi	Estimated Lowest Bearing Capacity, psi
Colorado	350	108-144	84	Shale	858	490
Utah	1000-2000	65-80	82	Shale	>6000	>5000
Illinois	660	108	84	2' coal over fireclay	>4820	2970
Virginia	1350-2000	52	52	Hard fireclay	>5000	>4000
West Virginia(1)	500-800	50	50	6" shale over fireclay	3420	1600
West Virginia(2)	400-800	60	60	Shale	3470	2600
West Virginia(3)	400-600	80	72	Shale	3440	2800
Pennsylvania(1)	440-600	58	54	6" shale over fireclay	5000	3100
Pennsylvania(2)	500-800	54	50	6" coal over fireclay	2890	2000

Consensus: McIntyre Bearing Capacities. There should be no major problems with any of the roof or base floor rocks relative to bearing capacity. Mining is planned to take up to 10 feet of extraction just below the roof caprock. Review of numerous exploration holes by McINTYRE and other consulting personnel reveals that the soft, shear coal or blackjack may be at an average depth of at least 12 feet and possibly 13 feet below the caprock. With a maximum of 10 feet of extraction at least 2-3 feet of hard coal should be left over the blackjack. This thickness should be sufficient to prevent bearing capacity problems from occurring relative to the blackjack itself. Should, for any reason, direct bearing on the blackjack occur by the longwall support bases, it is anticipated that major problems will occur. These problems will be related to the rapid penetration of the supports and their removal from the soft, shear coal. Of further critical importance is the necessity to insure that the longwall support manufacturers properly design the support bases. These bases should be such that not more than about 400-500 psi bearing is made on the coal floor under the maximum anticipated support load densities.

LONGWALL SUPPORT DESIGN

Caving Height. The height to which the immediate roof will extend or, in other words, the height of caving, will depend on the extraction height and the bulking factors of the immediate roof materials. Using the detailed geotechnical logs with appropriate bulking factors, the probable caving height may be calculated with the following formula[5]:

$$H_T = D_1 + D_2 + \ldots + \frac{h_m + D_1(1 - BF_1) + D_2(1 - BF_2) + \ldots}{BF_n - 1} \quad \{9\}$$

where
H_T = caving height
$D_1, D_2, \ldots$ = individual strata thickness
h_m = extraction height
$BF_1, BF_2, \ldots$ = individual strata bulking factor
BF_n = uppermost strata bulking factor

All roof rock types have been considered to be either sandstones, siltstones, claystones or combinations of the three. Based on previous Rocky Mountain Coalfield analyses, the bulking factors have been assumed as follows: sandstone - 1.20; siltstone - 1.25; claystone - 1.30. Applying Equation {9} to each of the materials found in each drill hole yields the caving height values shown in Table VIII. The average caving height is approximately 39 feet with a standard deviation of 4 feet. For analysis purposes, the caving height will be assumed to typically be 40 feet or four times the extraction height.

Caving Characteristics. Using the rock materials found in each of the boreholes as a guide, it is anticipated that for situations such as those found in Hole C81-22 and Hole C81-29 no hangup problems will develop. For situations similar to those found in Hole C81-23 and Hole C81-24 there may be some minor hangup problems at most. Observations in the No. 9A Mine do not indicate any definite problems. Therefore, it is concluded that a reasonably good cave should develop in the proposed panels of the No. 9G-4 Mine.

Average Rock Bulk Density. The average or typical rock bulk density of the roof materials is necessary for computing support loading magnitudes. One method of computing the roof bulk density is to use a weighted average formula such as the following:

$$\bar{\gamma} = \frac{D_1(\gamma_1) + D_2(\gamma_2) + \ldots + D_{nx}(\gamma_n)}{H_T} \qquad \{10\}$$

$\bar{\gamma}$ = weighted average bulk density
$D_1, D_2, \ldots$ = individual strata thickness
$\gamma_1, \gamma_2, \ldots$ = individual strata bulk density
D_{nx} = uppermost strata thickness
γ_n = uppermost strata bulk density
H_T = caving height

Applying Equation {10} to the four boreholes† would yield, in order, weighted average bulk densities of 160.4 pcf, 163.0 pcf, 162.6 pcf and 158.2 pcf. It will be assumed that the typical roof rock bulk density is the average of these four values or 161.0 pcf.

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† Assumes that the individual rock bulk densities shown in Table IV are applicable.

TABLE VIII

THEORETICAL CAVING HEIGHTS FOR BOREHOLE STRATA

HOLE	Number of Individual Strata Units	Caving Height(H_T),ft.
C81-22	2	37.1
C81-23	5	41.9
C81-24	3	41.6
C81-29	2	33.6

Support Load Density(USBM Method). The USBM method[6] assumes that the immediate roof behaves like a cantilever beam. The static load is then calculated and can be converted to a support load density by considering a specific shield size. The static load formula may be stated as follows:

$$W = L\,S\,H_T\,\bar{\gamma} \qquad \{11\}$$

where L = immediate roof length
S = supported width per shield
H_T = caving height
$\bar{\gamma}$ = immediate roof bulk density

The immediate roof length may be determined by the following formula:

$$L = l_o + l_s + l_u + l_w + h_m \qquad \{12\}$$

where l_o = overhang length(cave side)
l_s = shield length
l_u = unsupported roof width(tip side)
l_w = web thickness
h_m = extraction height

Using a Dowty 4-leg 610 US ton shield support as a model, the value of the immediate roof length would be

$$L = 10 + 12 + 1.2 + 3 + 10 = 36.2 \text{ ft.}$$

With the shield supports placed on 5 foot centers the total static load would be

$$W = (36.2)(5)(40)(161)$$

$$W = 583 \text{ tons}$$

The support load density on the Dowty shield would be

$$SLD = \frac{W}{A_c} \qquad \{12\}$$

where SLD = support load density
W = static load
A_c = canopy area

or

$$SLD = \frac{583}{(12.04)(4.67)}$$

$$SLD = 10.4 \text{ Tsf}$$

This support load density is considered relatively conservative owing to the assumptions used to determine the immediate roof length. Therefore, this value of support load density should be considered to represent the probable maximum that could occur. The resulting floor bearing pressure should be examined to determine whether or not it is acceptable. The bearing pressure may be computed as follows:

$$B_p = \frac{W}{A_b} \qquad \{13\}$$

where B_p = floor bearing pressure
W = static load
A_b = base area

or

$$B_p = \frac{583\ (2000)}{(2)(20.67)(86)} = 328 \text{ psi}$$

Owing to a bearing capacity in the upper hard coal of approximately 600 psi, floor failure problems should be minimal.

Support Load Density(Wilson's Method). The general method[7] of Wilson assumes that the roof may be modeled as blocks acting on the front and rear legs of the powered support. The loads on the legs are combined and a mean load density(MLD) is determined. Depending on the angle of roof cave(α) that is assumed, the MLD values can vary quite widely. For very weak roofs, such as those found in the U.K., the value of α is generally taken as 90°. The harder, stiffer beds found in the USA and Canada dictate that lesser cave angles be used. For purposes of this study, cave angle values of 45°, 55°, 65° and 75° will be examined. Applying the general method to the No. 4 Seam and using a Dowty 4-leg 610 US ton shield support as a model would yield the mean load densities shown in Table IX. The corresponding floor bearing pressures are also shown. For the Grande Cache area it is recommended that a cave angle on the order of 45°-55° be considered applicable for the Wilson method. Consequently, a mean load density between 7.5 and 8.0 tons/ft^2 would result, giving a floor bearing pressure of around 265 psi.

Longwall Support Design Summary. Based on the four boreholes that were examined, the following are believed to be typically representative:

Caving Height:	**40 ft.**
Caving Characteristics:	**Good**
Roof Bulk Density:	**161 pcf**
Maximum Support Load Density:	**10 - 11 Tsf**
Maximum Floor Bearing Pressure:	**330 psi**
Mean Load Density:	**7.5 - 8.0 Tsf**
Mean Floor Bearing Pressure:	**265 psi**

TABLE IX

SUPPORT LOAD DENSITIES: WILSON METHOD

Cave Angle	Total Load, tons	Mean Load Density, Tsf	Floor Bearing Pressure, psi
45°	532	8.9	300
55°	410	6.8	230
65°	314	5.2	180
75°	250	4.2	140

ENGINEERING AND RESEARCH RECOMMENDATIONS

In addition to the analyses and studies that have been undertaken to date for the proposed longwall panels in the No. 9G-4 Mine, several other geotechnical items should be addressed. A summary of these items is as follows:

1 - *CHAIN PILLAR DESIGN.* Coal pillars have previously been designed and successfully used in the Grande Cache mines. These pillars were designed for room and pillar mining methods and the corresponding loading which occurs during mine development and retreat. With the advent of longwall mining, the chain pillars created in the two-entry development system will be subject to very different loading than those used in room and pillar areas. For this reason detailed rock mechanics pillar design methods should be employed including (a) Wilson's confined core method and (b) probabilistic methods. It is recommended that such pillar design be undertaken and completed prior to beginning development of the first panel.

2 - *DRILLING AND STRENGTH TESTING.* During development work in the entries, it is recommended that drill cores be taken in the roof and floor of the gate entries. The cores should be analyzed and tested for strength parameters in order to have a preview of roof and floor conditions in advance of the longwall face.

3 - *SUBSIDENCE CONSIDERATIONS.* The potential surface subsidence should be preliminarily modeled so as to ascertain any adverse developments. During panel mining, the surface subsidence should be monitored so that the subsidence model can be accurately modified for actual conditions. An accurate and calibrated subsidence model will allow great versatility in optimally locating and mining longwall panels for the long term.

4 - *DISPLACEMENT/STRESS MONITORING.* Stress changes, pillar loading and entry closure should be monitored in the proposed gate entries. Such data would serve as a field measure of the mining conditions and provide insight into any potentially adverse developments.

CONCLUSIONS

The following conclusions are made relative to the proposed longwall panels in the No. 9G-4 Mine:

1 - *Floor Bearing Capacity* - The bearing capacity magnitude in the hard upper coals of the No. 4 Seam is low, but may be acceptable with properly designed longwall supports. The bearing capacity of the underlying soft, shear coal or blackjack is much too low to be used as an immediate floor in a longwall panel.

2 - *Support Load Density* - The immediate roof is expected to be on the order of 40 feet thick and reasonably good caving is expected. The maximum support load density may be on the order of 10 - 11 Tsf, but the design level, or mean load density, should be about 7.5 - 8.0 Tsf. Corresponding bearing pressures on the floor may be up to 330 psi, but should typically be around 265 psi, depending on the shield support model.

3 - *Ongoing Engineering and Research* - Recommended items for further geotechnical evaluation include

a - chain pillar design
b - drilling and strength testing
c - subsidence considerations
d - displacement/stress monitoring

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APPENDIX I

DETAILED GEOTECHNICAL LOGS

GEOTECHNICAL CORE LOGS - DATA PRESENTATION

A geotechnical core log has been prepared for portions of the materials in the borehole. Presentation of the graphic logs and corresponding descriptive logs is as follows:

1. *Definition of Technical Terms* - Some of the technical terms used in the descriptive geotechnical logs may be unfamiliar to the reader. Therefore, definitions of the more unfamiliar terms are presented.
2. *Key to Graphic Geotechnical Logs* - The key to the graphic descriptions of the geotechnical logs is presented in Figure I-1. This key describes the meaning of the symbols used in the graphic logs.
3. *Graphic Geotechnical Logs* - The graphic log of all the materials in the hole is presented first.
4. *Descriptive Geotechnical Logs* - The detailed descriptive log for the hole follows the graphic log.

TECHNICAL TERMS: DESCRIPTIVE GEOTECHNICAL LOGS

HARDNESS

The hardness or consistency of the material is recorded using a visual and sensory description. It is assumed that the difference between a soil and rock lies in the question of hardness or strength. The hardest soil is a "very stiff soil" and this borders on the softest rock which is classified as a "very soft rock". Five descriptions of soil consistency or hardness have been devised ranging from "very soft" to "very stiff" soils. Likewise, five descriptions of rock hardness have been similarly drawn up, based on simple field tests such as behavior when scratched with a knife or hit with a geological hammer. The rock descriptions range from "very soft rock" to "very, very hard rock". The soils thus merge into rocks, all on a basis of hardness and the whole 10-point scale ranges from "very soft soil" to "very, very hard rock". The categories for hardness of the material are recorded according to the following designations:

	Description	Code
Very soft soil	- easily molded in fingers; shows distinct heelmarks	0
Soft soil	- molds in fingers with strong pressure; faint heelmarks	1
Firm soil	- very difficult to mold in fingers; difficult to cut with a hand spade	2
Stiff soil	- cannot be molded in fingers; cannot be cut with hand spade and requires hand-picking for excavation	3
Very stiff soil	- very tough and difficult to move with handpick; requires pneumatic spade for excavation	4

Very soft rock	- material crumbles under firm blows with a sharp end of a geological pick and can be peeled off with a knife. It is too hard to cut a triaxial sample by hand	5
Soft rock	- can just be scraped and peeled with a knife; indentations 1/16 in. to 1/8 in. show in the specimen with firm blows of the pick point	6
Hard rock	- cannot be scraped or peeled with a knife; hand-held specimen can be broken with a hammer end of a geological pick with a single firm blow	7
Very hard rock	- hand-held specimen breaks with hammer end of pick under more than one blow	8
Very, very hard rock	- specimen requires many blows with geological pick to break through intact material	9

ALPHA (α) ANGLE

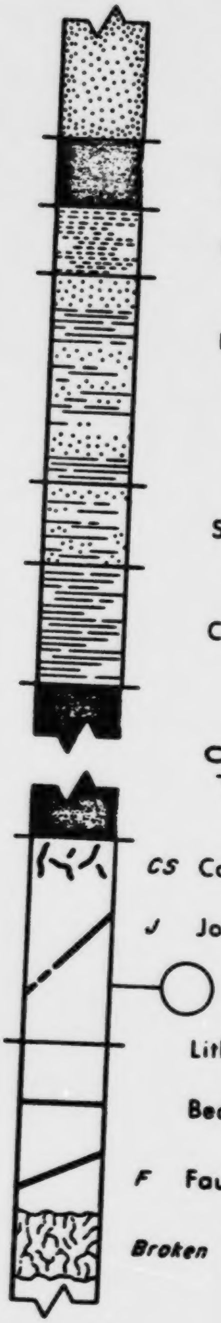
The α angle is the minimum angle of intersection between the structure or discontinuity and the core (or drill hole) axis. The magnitude of α varies from 0° to 90°.

RQD

The RQD of core means the Rock Quality Designation of that core interval in question. RQD is effectively a modified core recovery. To compute the RQD for any core interval, the number of pieces of core 4 inches or more in length are cumulatively measured and divided by the length of the actual core interval drilled. In other words, RQD is computed in the same procedure as the standard core recovery, except all pieces 4 inches or less are assumed to be lost. RQD magnitudes accordingly range from 0% to 100%.

FIGURE I-1

KEY FOR GRAPHIC GEOTECHNICAL LOGS

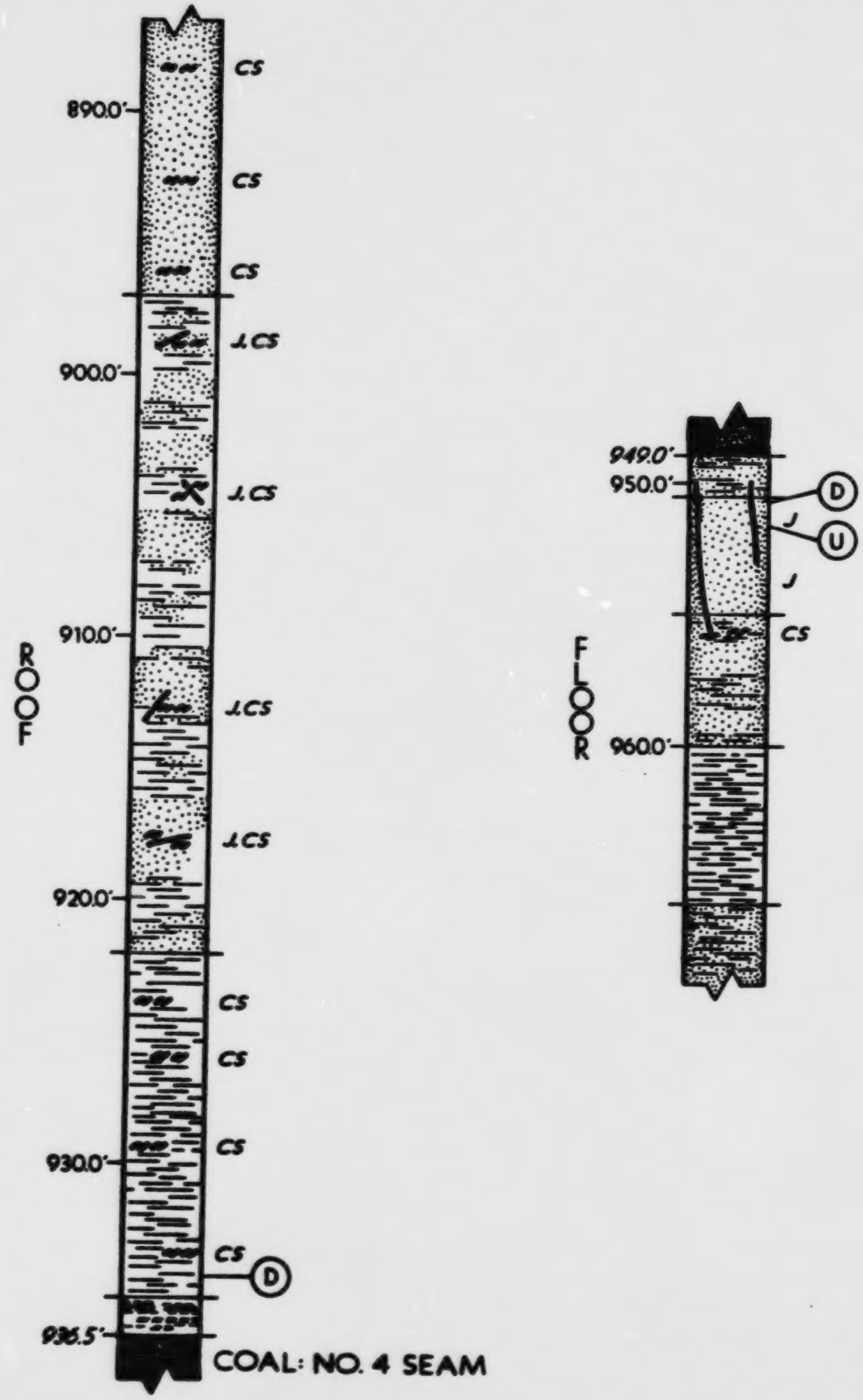


ROCK TEST TYPE

Ⓤ	UNIAXIAL COMPRESSIVE, MODULII
Ⓣ	TRIAXIAL
Ⓕ	FLEXURAL
Ⓑ	BRAZILIAN TENSILE
Ⓓ	DIRECT SHEAR
Ⓖ	DENSITY, % MOISTURE, POROSITY
(S_L)	SLAKE POTENTIAL
(S_W)	SWELL POTENTIAL
(V_P)	P-WAVE VELOCITY

FIGURE I-2

GEOTECHNICAL LOG OF ROOF AND FLOOR OF NO. 4 SEAM
DRILL HOLE C81-22



DRILL HOLE NO. C81-22

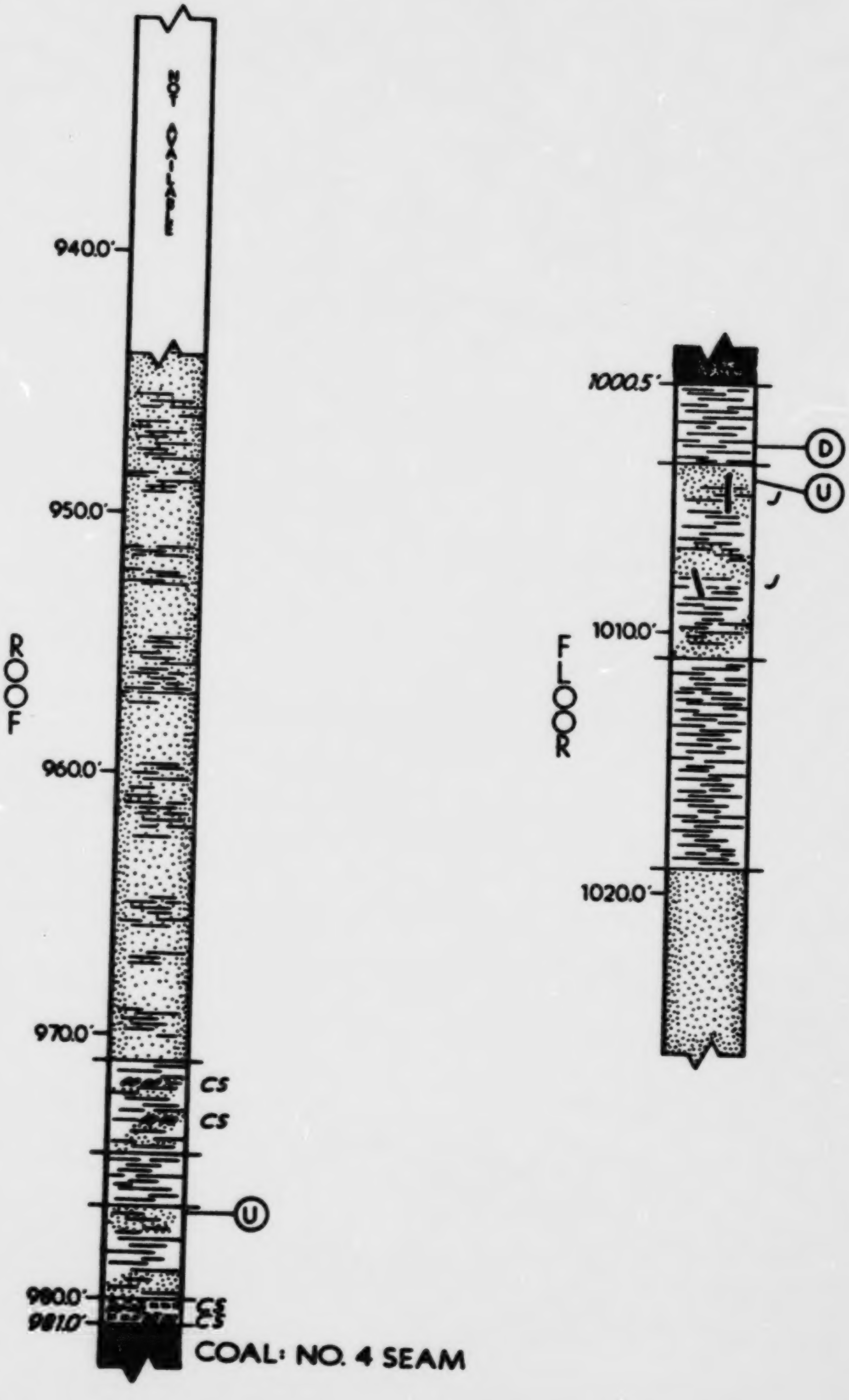
DEPTH, FT.	GEOTECHNICAL DESCRIPTION
881.0-897.0	SANDSTONE: Medium-grained; crossbedded; well indurated; hardness 8; occasional tightly compacted smoothly slickensided bedding planes; widely spaced open joints or bedding surfaces showing slickensides and calcite infilling material; estimated compressive strength 10,000 psi.
897.0-922.0	SANDSTONE, SILTSTONE and CLAYSTONE: Thinly to coarsely interbedded; generally upwards fining;; slightly carbonaceous in fine-grained intervals; some coaly debris; crossbedded; hardness 8-9; open bedding breaks and joint surfaces generally along smooth slickensided surfaces healed with calcite; joints closely to moderately closely spaced; dense and competent unit; estimated high RQD; estimated strength 10,000-20,000 psi; gradational contacts.
922.0-935.0	CLAYSTONE: Slightly silty; grainsize fining downwards; carbonaceous content increasing at depth; well indurated; fairly homogeneous; hardness 6-7; possible scattered slickensides; fairly competent; probably high RQD; estimated strength 4000-5000 psi; gradational contacts.
935.0-936.5	CARBONACEOUS CLAYSTONE: Silty with grainsize fining downwards; coaly; fissile; dense; well indurated; hardness 6; brittle; abundant slickensides along bedding laminae; probably incompetent; calcite locally along bedding planes; low RQD; estimated strength 3000 psi.
936.5-949.0	COAL: No. 4 Seam; removed.
949.0-950.5	SANDSTONE and SILTSTONE: Thinly interbedded; minor coaly debris; well indurated; competent; hardness 7; high RQD; estimated strength 8000 psi; gradational lower contact.
950.5-955.0	SANDSTONE: Medium-grained; occasional thin silty-clayey bands; occasional steep joint surfaces healed with calcite; well indurated; hardness 8; gradational lower contact; high RQD; estimated strength 8,000-12,000 psi.
955.0-960.0	SANDSTONE and SILTSTONE: Thinly interbedded; low angle crossbedding; well indurated; hardness 7; high RQD; estimated srength 8,000-10,000 Osi; fairly sharp lower contact.

DRILL HOLE NO. C81-22 (CONTINUED)

DEPTH, FT.	GEOTECHNICAL DESCRIPTION
960.0-966.0	CLAYSTONE: Silty; slightly carbonaceous; grainsize fining downwards; fairly homogeneous; fissile in basal 2'; well indurated; fairly competent; medium to high RQD; estimated strength 3000-5000 psi; lower contact sharp and unbroken.
966.0-969.0	SILTSTONE: Sandy; grainsize fining upwards; well indurated; hardness 7; strong sandstone units at depth; estimated 100% RQD; estimated strength 8,000-12,000 psi.

FIGURE I-3

GEOTECHNICAL LOG OF ROOF AND FLOOR OF NO. 4 SEAM
DRILL HOLE C81-23



DRILL HOLE NO. C81-23

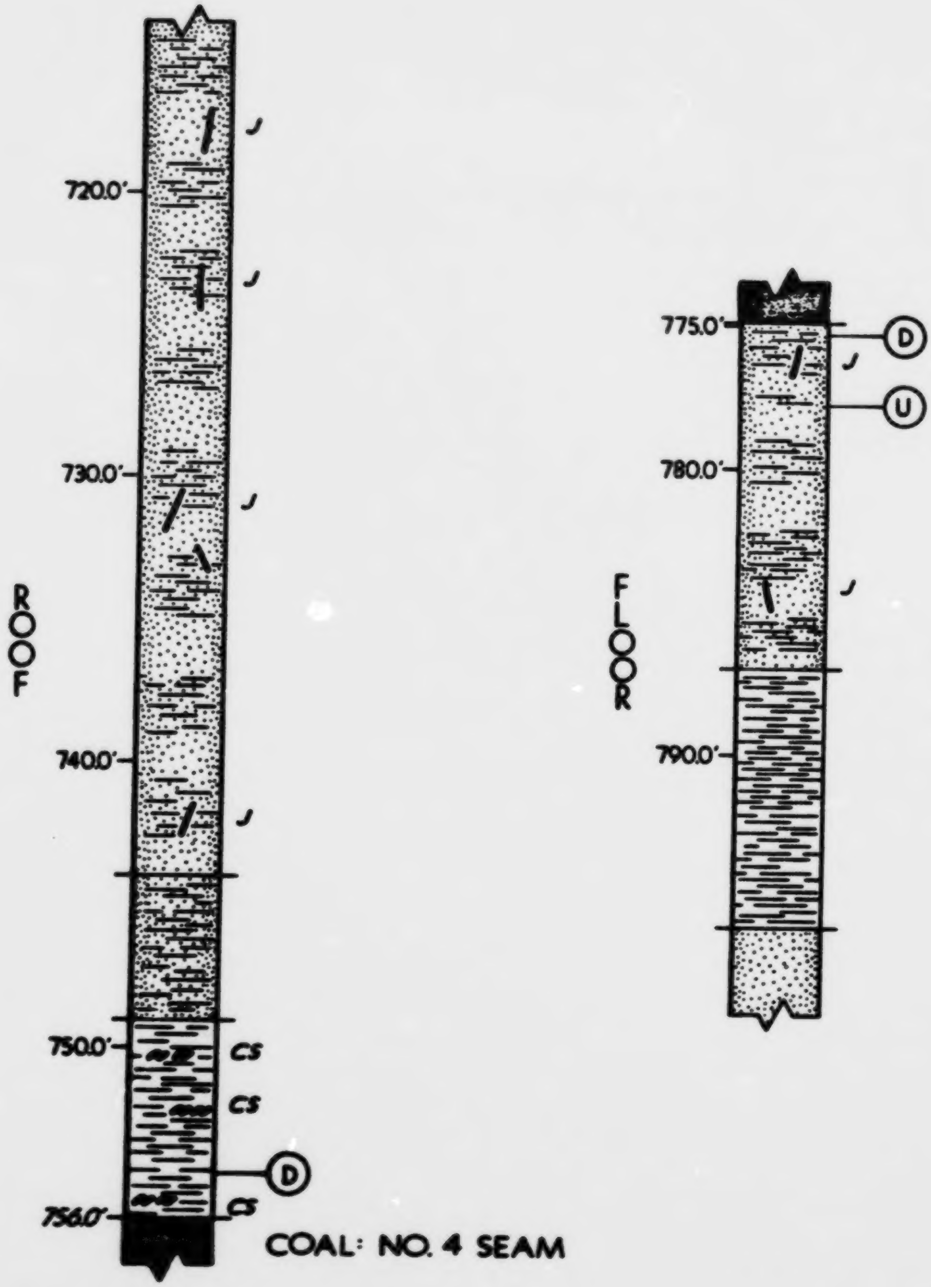
DEPTH, FT.	GEOTECHNICAL DESCRIPTION
944.0-971.0	SANDSTONE and SILTSTONE: Thinly interbedded; slightly carbonaceous within fine-grained intervals; low angle crossbedding and minor convolute bedding; well indurated; hardness 7-8; breaks fairly smoothly along carbonaceous bedding planes; no joints observed; estimated strength of 8,000-10,000 psi; gradational lower contact.
971.0-974.5	SILTSTONE and CLAYSTONE: Thinly interbedded; slightly carbonaceous with abundant coaly debris along bedding laminae; occasional slickensides; well indurated; hardness 7; breaks easily and fairly smoothly along bedding laminae; RQD probably high; fairly competent; gradational contacts; estimated strength 5,000-8,000 psi.
974.5-976.5	CLAYSTONE: Abundant thin silty bands; slightly carbonaceous; thinly scattered coaly debris; well indurated; hardness 7; competent; gradational contacts; estimated strength 5000 psi.
976.5-980.0	SILTSTONE and CLAYSTONE: Thinly interbedded; slightly carbonaceous; well indurated; hardness 7; breaks fairly roughly, but easily along clayey bedding planes; competent; sharp broken lower contact; estimated strength 5000-8000 psi.
980.0-981.0	CARBONACEOUS CLAYSTONE: Coaly; fissile; well indurated; hardness 6; abundant low angle slickensides; incompetent; appears to be adversely affected by atmospheric conditions; estimated strength of intact rock 3000 psi.
981.0-1000.5	COAL: No. 4 Seam, removed.
1000.5-1003.5	CLAYSTONE: Silty, including thin silty bands; carbonaceous; abundant coaly debris; well indurated; hardness 6-7; breaks easily along carbonaceous bedding planes; competent; gradational lower contact; estimated strength 5000-6000 psi.
1003.5-1011.0	SANDSTONE, SILTSTONE and CLAYSTONE: Thinly interbedded; slightly carbonaceous where fine-grained; low angle cross-bedding; well indurated; hardness 7; closely spaced tight healed vertical joints with calcite infilling material; breaks fairly smoothly along clayey bedding planes; fairly sharp unbroken lower contact; estimated strength 6000-9000 psi.

DRILL HOLE NO. C81-23 (CONTINUED)

DEPTH, FT.	GEOTECHNICAL DESCRIPTION
1011.0-1019.0	CLAYSTONE: Minor thin silty bands decreasing at depth; generally homogeneous; friable; well indurated; hardness 6; moderate slake potential; fairly competent; breaks roughly along bedding laminae; sharp unbroken lower contact; estimated strength 3000-5000 psi; estimated moderate RQD, although adversely affected by drying conditions.
1019.0-1026.0	SANDSTONE: Fine-grained; thin slightly carbonaceous silty bands; very well indurated; hardness 8; breaks smoothly along silty bands; high estimated RQD; estimated strength 10,000-15,000 psi.

FIGURE I-4

GEOTECHNICAL LOG OF ROOF AND FLOOR OF NO. 4 SEAM
DRILL HOLE C81-24

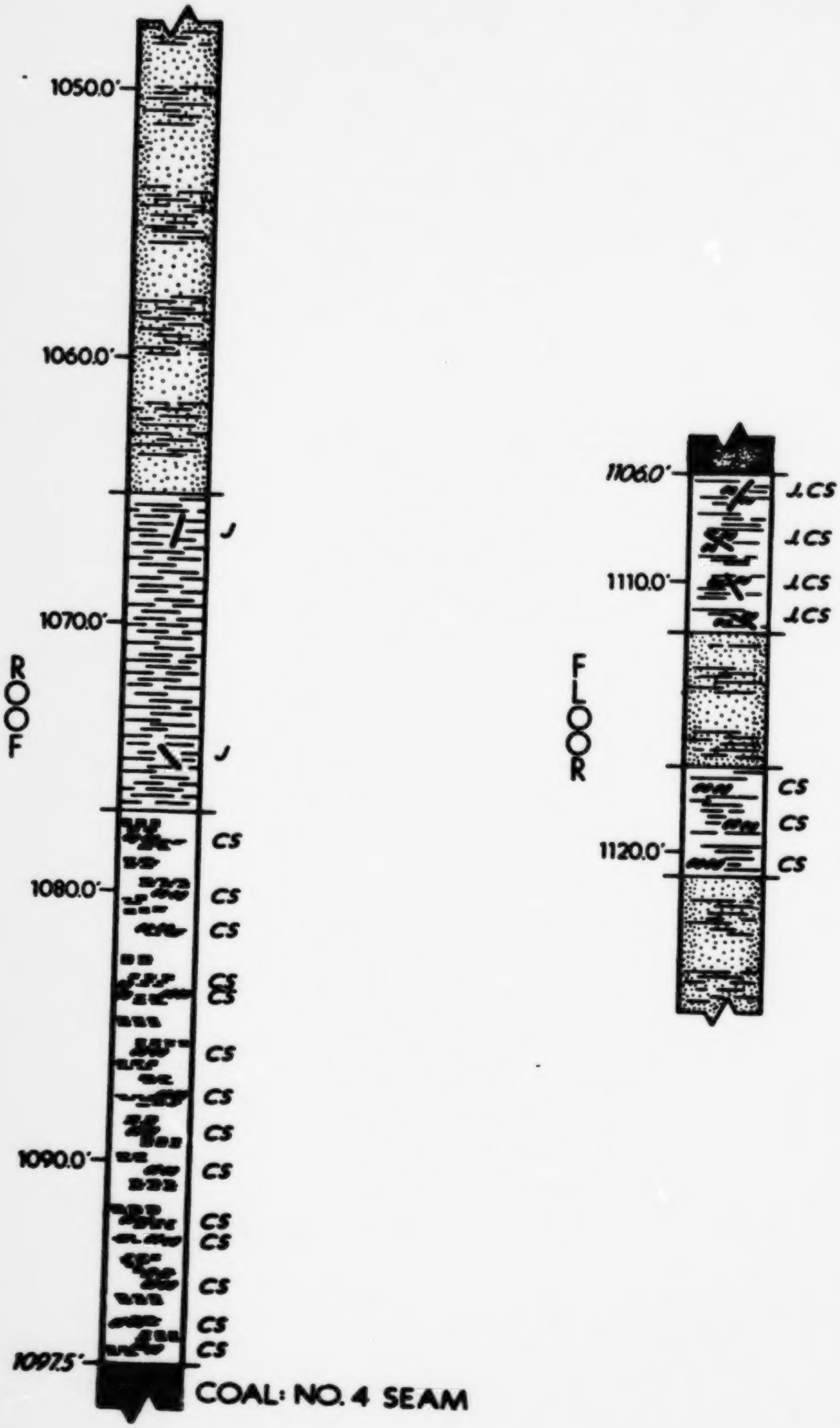


DRILL HOLE NO. C81-24

DEPTH, FT.	GEOTECHNICAL DESCRIPTION
714.0-744.0	SANDSTONE and SILTSTONE: Thinly interbedded; slightly carbonaceous in fine-grained intervals; very well indurated; hardness 7; breaks smoothly and easily along bedding laminae; moderately closely to widely spaced tight healed joints with calcite infilling material; low angle crossbedding and faint convolute bedding; muddier in basal 6'; estimated strength 8,000-10,000 psi; probable high RQD; may cave well but in larger blocks.
744.0-749.0	SILTSTONE: Sandy locally; slightly carbonaceous; well indurated; hardness 7; scattered coaly debris; breaks roughly and easily along bedding laminae; bedding structures disorganized; no joints or other structures observed; gradational unbroken contacts; estimated strength 6000-7000 psi; may cave well in larger blocks.
749.0-754.0	CLAYSTONE: Silty upper contact; grainsize fining downwards; more coaly downwards; occasional slickensides; well indurated; hardness 7; breaks roughly along bedding laminae; slightly friable; strong; gradational contacts; estimated strength 4000 psi; estimated RQD 50%.
754.0-756.0	CLAYSTONE: Coaly; slickensided; well indurated; hardness 6; fair strength estimated at 3000 psi; core below removed.
756.0-775.0	COAL: No. 4 Seam, removed.
775.0-787.0	SANDSTONE and SILTSTONE: Thinly interbedded; low angle crossbedding and convolute bedding; rare steep tight joints with calcite infilling material; well indurated; hardness 8; breaks fairly smoothly along carbonaceous bedding planes; high RQD; gradational lower contact; estimated strength 10,000 psi.
787.0-796.0	CLAYSTONE: Carbonaceous; slightly silty; homogeneous; well indurated; strong; hardness 6-7; gradational contacts; estimated strength 4000 psi.
796.0-	SANDSTONE: Well indurated; exceedingly strong; this unit persists below claystone contact at 796.0 feet.

FIGURE I-5

GEOTECHNICAL LOG OF ROOF AND FLOOR OF NO. 4 SEAM
DRILL HOLE C81-29



DRILL HOLE NO. C81-29

DEPTH, FT.	GEOTECHNICAL DESCRIPTION
1036.0-1042.0	SANDSTONE: Medium-grained; faint bedding plane structures; massive; homogeneous; widely spaced; smooth joint surfaces with calcite infilling material; hardness 8-9; sharp, broken lower contact; RQD 100%; estimated strength 15,000 psi.
1042.0-1065.0	SANDSTONE and SILTSTONE: Thinly interbedded; slightly carbonaceous; well indurated; hardness 7-8; gradational lower contact; no apparent structural influences; fairly high RQD; estimated strength 8,000 psi.
1065.0-1077.0	CLAYSTONE: Silty; grainsize fining downwards; strong; well indurated; hardness 6-7; breaks roughly and easily along bedding planes; clean tight joint surfaces widely spaced; gradational contacts; estimated high RQD; estimated strength 4000-7000 psi.
1077.0-1097.5	CARBONACEOUS CLAYSTONE: Slightly carbonaceous locally to very coaly; local shear zones comprise substantial portion of this unit; strongly slickensided throughout; well indurated; hardness 5-6; very weak; very low RQD; estimated strength of intact materials 2500-3500 psi.
1975.5-1106.0	COAL: No. 4 Seam, removed.
1106.0-1112.0	CLAYSTONE: Silty; grainsize increasing downwards; friable; well indurated; hardness 6; slickensided and jointed throughout; incompetent; low RQD; breaks easily and roughly along bedding planes; estimated strength 3000 psi.
1112.0-1117.0	SANDSTONE and SILTSTONE: Thinly interbedded; abundant coaly debris; well indurated; hardness 7; breaks with difficulty along coaly, clayey zones; strong; high RQD; estimated strength 10,000 psi.
1117.0-1121.0	CLAYSTONE: Silty; scattered coaly debris; well indurated; hardness 6; scattered slickensides; fair competence; fairly high RQD; sharp broken contacts; estimated strength 4000-6000 psi.
1121.0-1126.0	SANDSTONE, SILTSTONE and CLAYSTONE: Mudstone composition; grainsize fining downwards; scattered coaly debris; competence decreasing downwards; well indurated; hardness 6-7; competent siltstones found at depth; estimated strength 3000-8000 psi increasing upwards; gradational lower contact.

APPENDIX II

DETAILED LABORATORY STRENGTH TESTS

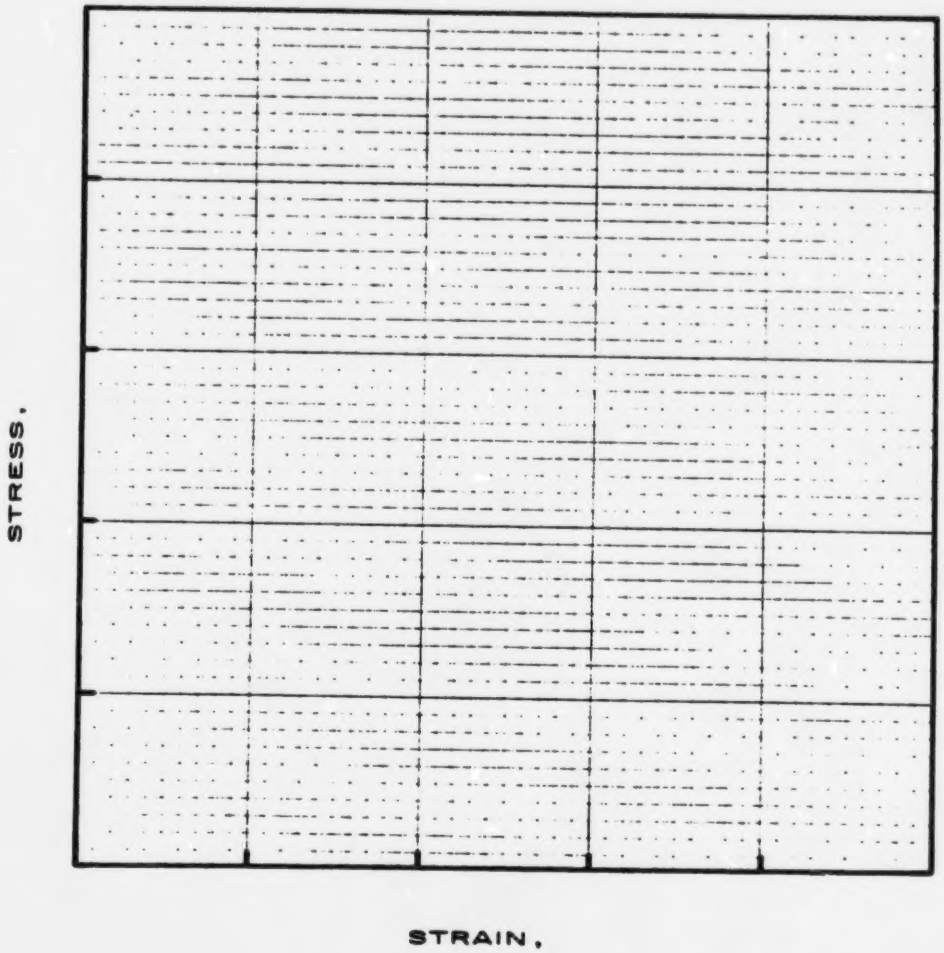
SEEGMILLER INTERNATIONAL
MINING GEOTECHNICAL CONSULTANTS / ENGINEERS

UNIAXIAL COMPRESSIVE STRENGTH AND ELASTIC MODULUS

Sample No. 9G-23R-1U Client McINTYRE Date 10/28/82

Sample Size 61.0mm dia. x 138.75mm Surface Area 2,920mm² Tested by RPS

PRE-TEST DESCRIPTION: SILTSTONE(Hole No. C81-23, 976.4'): Clayey locally to sandy; mudstone composition; slightly friable; well indurated; very minor edge losses; no apparent structures; good sample with good edges and end surfaces.



TEST COMMENTS: Very Good Test: Stepwise brittle failure; minor shear surface development near top end surface(α=10°-15°); several vertical cracks through length divided sample into several bladed pieces; minor bedding plane breaks.

Force at Failure 165 kN Uniaxial Compressive Strength 56.4 MPa

Tangent Modulus N/A Secant Modulus N/A

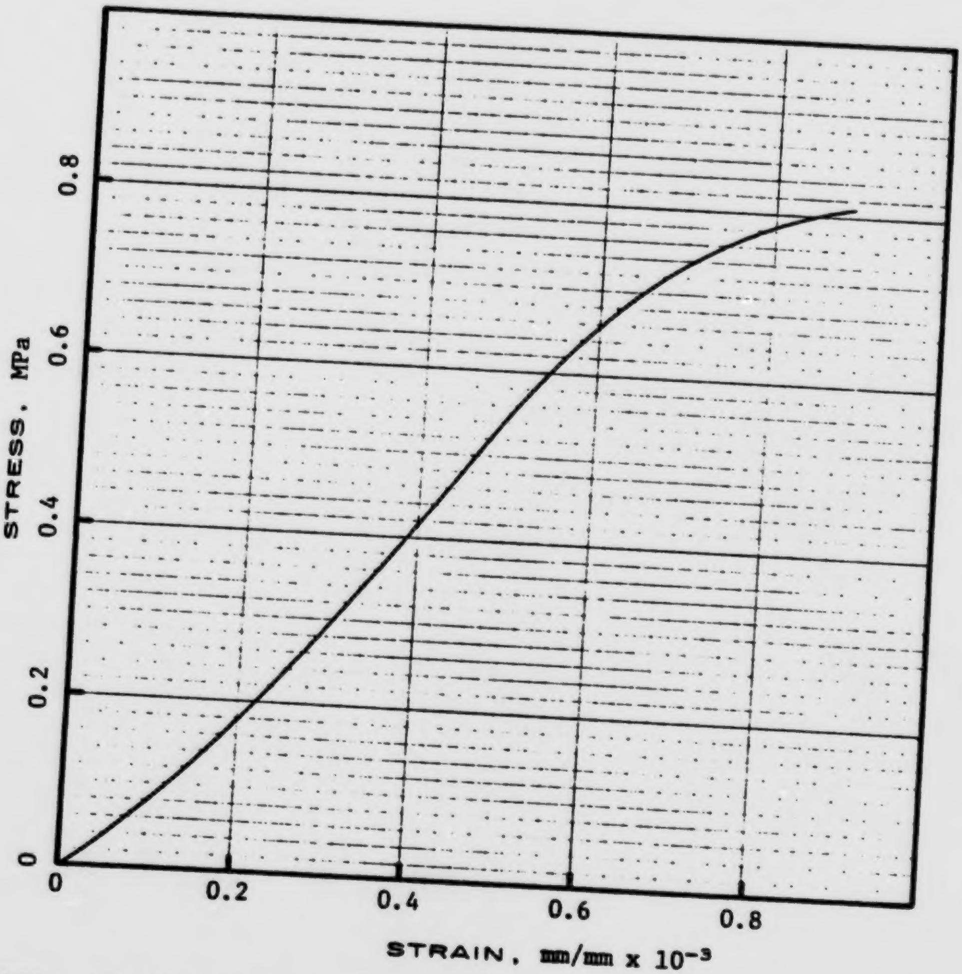
Modulus Ratio N/A Stiffness N/A Poisson's Ratio N/A

UNIAXIAL COMPRESSIVE STRENGTH AND ELASTIC MODULUS

Sample No. 9G-COAL-1U **Client** McINTYRE **Date** 11/01/82

Sample Size 32.55mm dia. x 59.2mm **Surface Area** 830mm² **Tested by** RPS&LES

PRE-TEST DESCRIPTION: COAL(From Lump Sample): Mostly dull; clean; tightly spaced cleat set oriented parallel to core axis; exceedingly brittle; only very minor losses along cleat surfaces on core perimeter; good sample with good edges and end surfaces.



TEST COMMENTS: Excellent Test: Stepwise crumbling failure; stepped and fairly planar shear surface formed dividing sample evenly from edge ot edge(α=30°); two pieces plus considerable small fragments.

Force at Failure 0.667 kN **Uniaxial Compressive Strength** 0.804 MPa

Tangent Modulus 1.11 GPa **Secant Modulus** N/A

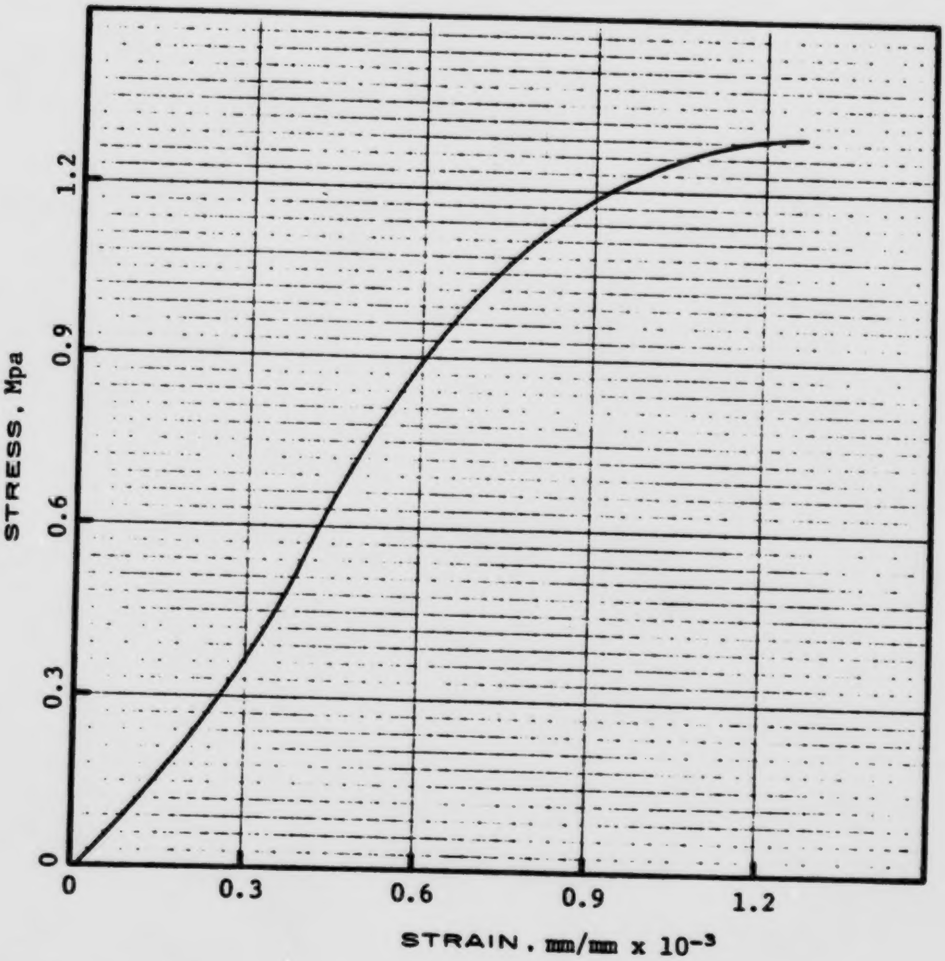
Modulus Ratio N/A **Stiffness** N/A **Poisson's Ratio** N/A

UNIAXIAL COMPRESSIVE STRENGTH AND ELASTIC MODULUS

Sample No. 9G-COAL-2U Client McINTYRE Date 11/01/82

Sample Size 32.5mm dia. x 56.8mm Surface Area 830mm² Tested by RPS&LES

PRE-TEST DESCRIPTION : COAL: Mostly dull; clean; exceedingly brittle; strong; tightly spaced cleat set oriented parallel to core axis; some loss on one end surface required refacing with Phenyl; good sample with good edges and end surfaces.



TEST COMMENTS : Excellent Test: Slow crumbling failure; two steep irregular conjugate shear surfaces-tension cracks formed; most of sample proceeded to crumble along cleat surfaces and bedding planes into a pile of rubble.

Force at Failure 1.07 kN Uniaxial Compressive Strength 1.29 MPa

Tangent Modulus 1.62 GPa Secant Modulus N/A

Modulus Ratio N/A Stiffness N/A Poisson's Ratio N/A

UNIAXIAL COMPRESSIVE STRENGTH AND ELASTIC MODULUS

Sample No. 9G-22F-U1 Client McINTRYE Date 10/28/82

Sample Size 61.15mm dia. x 164.2mm Surface Area 2,940mm² Tested by RPS

PRE-TEST DESCRIPTION : SANDSTONE(Hole No. C81-22; 951.6'): Fine-grained; disseminated carbonaceous debris; well indurated; vertical well healed microfracture passes through length of sample; good sample with very good edges and end surfaces.

STRESS,

STRAIN,

TEST COMMENTS : Very Good Test: Stepwise brittle failure; high angle shear development in upper 50mm(α=5°-10°); major crack divided sample along preexisting joint surface; other subvertical cracks in intact rock forming; two pieces plus minor small fragments.

Force at Failure 173 kN Uniaxial Compressive Strength 59.0 MPa

Tangent Modulus N/A Secant Modulus N/A

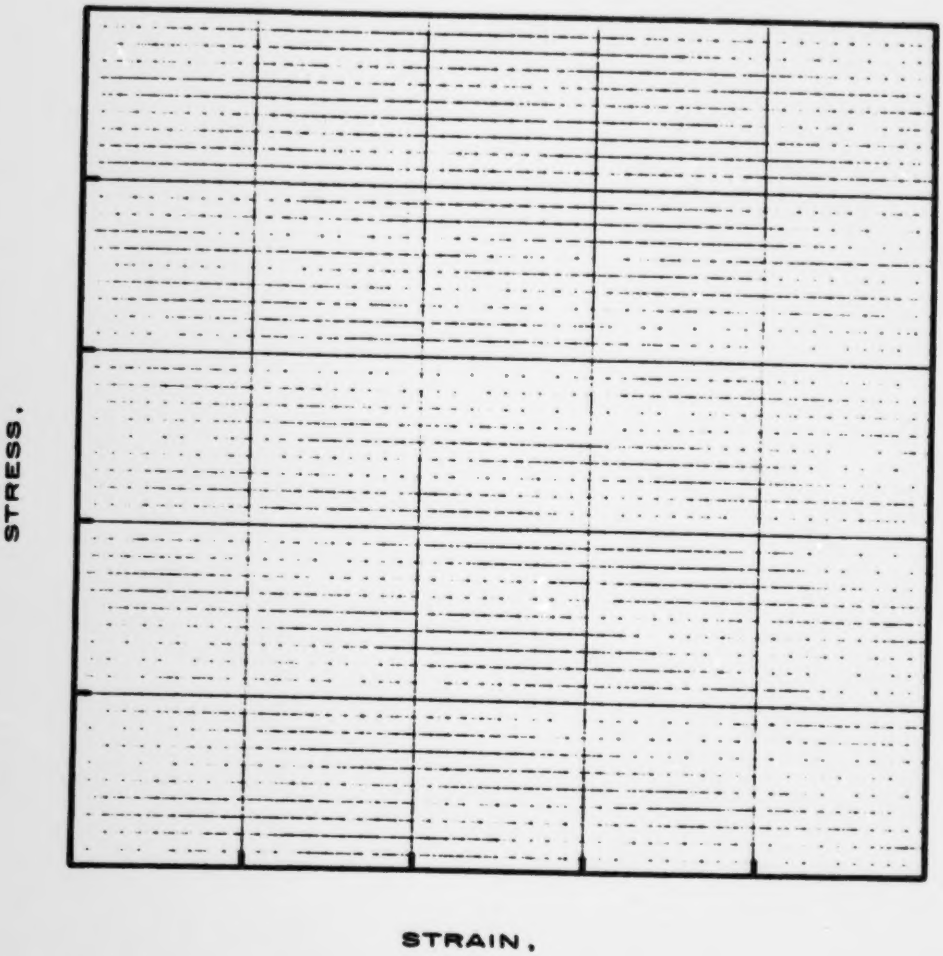
Modulus Ratio N/A Stiffness N/A Poisson's Ratio N/A

UNIAXIAL COMPRESSIVE STRENGTH AND ELASTIC MODULUS

Sample No. 9G-23F-U1 **Client** McINTYRE **Date** 10/28/82

Sample Size 61.05mm dia. x 168.95mm **Surface Area** 2,930mm² **Tested by** RPS

PRE-TEST DESCRIPTION : SANDSTONE(Hole No. C81-23; 1004.5'): Fine-grained to silty; carbonaceous flasers; well indurated; two parallel microfractures pass through most of sample length(α=8°); good sample with good edges and end surfaces.



TEST COMMENTS : Very Good Test: Fairly sudden brittle failure; minor local high angle shear development at ends; most cracks formed along preexisting structures(α=15°±); cracks also forming in intact rock; three bladed pieces plus fragments.

Force at Failure 218 kN **Uniaxial Compressive Strength** 74.4 MPa

Tangent Modulus N/A **Secant Modulus** N/A

Modulus Ratio N/A **Stiffness** N/A **Poisson's Ratio** N/A

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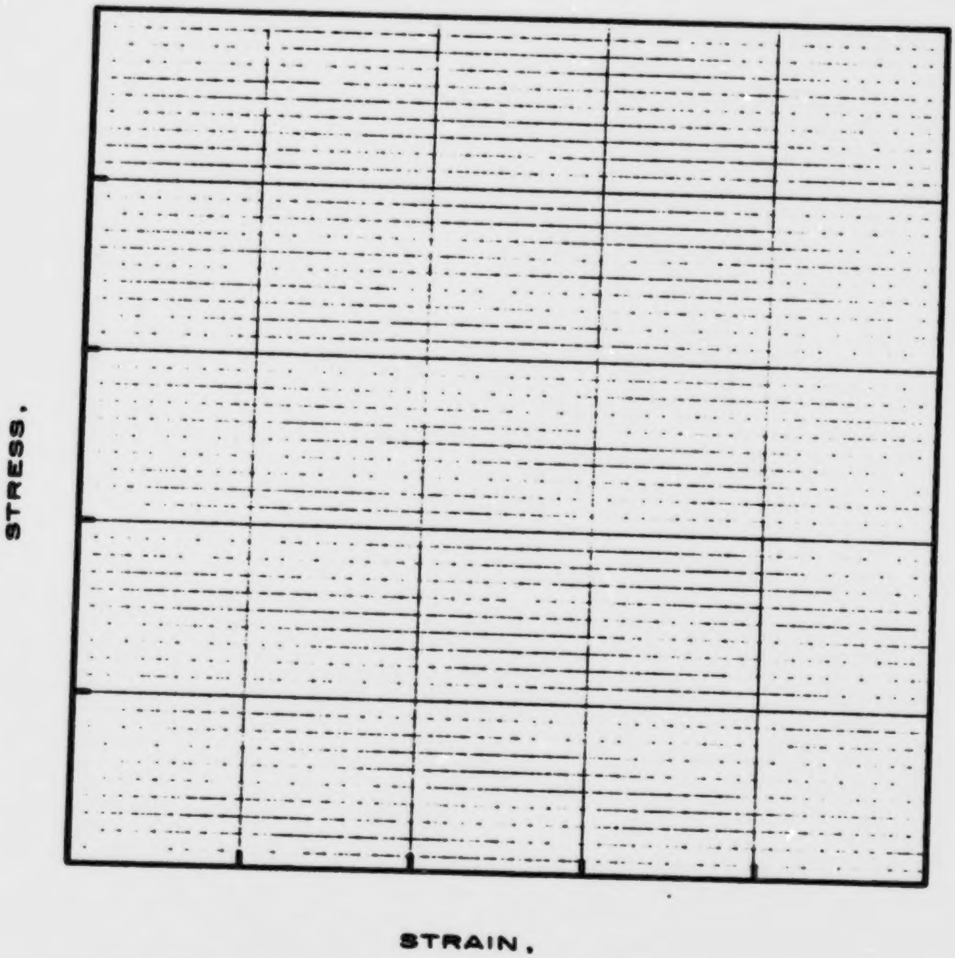
UNIAXIAL COMPRESSIVE STRENGTH AND ELASTIC MODULUS

Sample No. 9G-24F-U1 **Client** McINTYRE **Date** 10/28/82

Sample Size 60.8mm dia. x 151.5mm **Surface Area** 2,900mm² **Tested by** RPS

PRE-TEST DESCRIPTION: SANDSTONE(Hole No. C81-24; 777.6'): Fine-grained; disseminated carbonaceous debris; few clayey stringers; well indurated; no apparent structure; good sample with good edges and end surfaces.



TEST COMMENTS: Excellent Test: Stepwise brittle failure; minor shear surface formed at lower end(α=10°); several vertical cracks divided length into several bladed fragments; minor break along pre-existing joint surface near the bottom end.

Force at Failure 305 kN **Uniaxial Compressive Strength** 105 MPa

Tangent Modulus N/A **Secant Modulus** N/A

Modulus Ratio N/A **Stiffness** N/A **Poisson's Ratio** N/A

SEEGMILLER INTERNATIONAL
MINING GEOTECHNICAL CONSULTANTS / ENGINEERS

ROCK DIRECT SHEAR STRENGTH

SAMPLE NUMBER 9G-22R-1D

CLIENT McINTYRE
Date October 28, 1982
PROPERTY 9G Mine
By RPS
Hole Number C81-22
Hole Location 55,463' N 97,008' E
Hole Size HQ
Collar Elevation 5,055'
Location in Hole 934.4'
General Hole Bearing Vertical
General Hole Dip 90°

Sample Diameter 61.2mm
Force at Failure 25.4 kN
Strength 4.31 MPa

Pre-testing Sample Description CARBONACEOUS CLAYSTONE: Scattered thin coaly debris; well indurated; solid and intact; bedding planes oriented 68° to core axis; tends to split easily along bedding planes; good sample in good condition.

Post-testing Sample Description and Test Comments Good Test: Slow, stepwise failure; failure tended to follow bedding planes; central disc mostly intact with minor spalling; three pieces plus fragments.

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ROCK DIRECT SHEAR STRENGTH

SAMPLE NUMBER 9G-24R-1D

CLIENT McINTYRE Date October 28, 1982

PROPERTY 9G Mine By RPS

Hole Number C81-24 Hole Location 54,004' N 94,001' E

Hole Size HQ Collar Elevation 5,364' Location in Hole 752.7'

General Hole Bearing Vertical General Hole Dip 90°

Sample Diameter 60.8mm *Force at Failure* 38.9 kN *Strength* 6.71 MPa

Pre-testing Sample Description CARBONACEOUS CLAYSTONE: Thin coaly stringers; friable; well indurated; bedding planes oriented 70° to core axis; intact; good sample in good condition.

Post-testing Sample Description and Test Comments Excellent Test: Fairly sudden failure; central disc fairly well formed; crumbling of outer pieces as failure tended to follow bedding planes; three pieces plus fragments.

SEEGMILLER INTERNATIONAL
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ROCK DIRECT SHEAR STRENGTH

SAMPLE NUMBER 9G-22F-1D

CLIENT McINTYRE Date October 28, 1982

PROPERTY 9G Mine By RPS

Hole Number C81-22 Hole Location 55,463' N 97,008' E

Hole Size HQ Collar Elevation 5,055' Location in Hole 950.6'

General Hole Bearing Vertical General Hole Dip 90°

Sample Diameter 61.1mm *Force at Failure* 124 kN *Strength* 21.1 MPa

Pre-testing Sample Description SANDSTONE: Fine-grained; disseminated coaly debris; well indurated; two irregular discontinuous, well healed microfractures; no apparent bedding structures; good sample in good condition.

Post-testing Sample Description and Test Comments Excellent Test: Fairly sudden failure; well developed intact central disc; no modifications of failure mode along bedding planes; three pieces plus minor fragments.

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ROCK DIRECT SHEAR STRENGTH

SAMPLE NUMBER 9G-23F-1D

CLIENT McINTYRE

Date October 28, 1982

PROPERTY 9G Mine

By RPS

Hole Number C81-23 Hole Location 54,901' N 95,932' E

Hole Size HQ Collar Elevation 5,265' Location in Hole 1002.8'

General Hole Bearing Vertical General Hole Dip 90°

Sample Diameter 61.0mm *Force at Failure* 61.6 kN *Strength* 10.5 MPa

Pre-testing Sample Description CLAYEY SILTSTONE: Carbonaceous; fairly homogeneous; well indurated; bedding planes oriented 80° to core axis; tends to separate roughly along bedding planes; good sample in good condition.

Post-testing Sample Description and Test Comments Excellent Test: Stepwise brittle failure; shear surfaces fairly well developed aroung well developed central disc; three pieces plus fragments.

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ROCK DIRECT SHEAR STRENGTH

SAMPLE NUMBER 9G-24F-1D

CLIENT McINTYRE Date October 28, 1982

PROPERTY 9G Mine By RPS

Hole Number C81-24 Hole Location 54,004' N 94,001' E

Hole Size HQ Collar Elevation 5,364' Location in Hole 775.4'

General Hole Bearing Vertical General Hole Dip 90°

Sample Diameter 60.7mm *Force at Failure* 117 kN *Strength* 20.2 MPa

Pre-testing Sample Description SILTSTONE: Clayey; thin carbonaceous bedding laminae; one end surface is a slickensided clayey bedding plane; well indurated; exceedingly strong general appearance; bedding planes oriented 70° to core axis; good sample in good condition.

Post-testing Sample Description and Test Comments Good Test: Stepwise failure; minor shear surface development oblique to bedding planes; ultimate failure occurred by splitting along bedding planes; central disc experienced brittle fragmentation; six pieces of variable size formed.

POINT LOAD TEST DATA

		AVERAGE VALUES, MPA		
SOURCE	DESCRIPTION	I(AXIAL)	I(DIAMETRAL)	I(ANISOTROPY)
C81-22	Sandy Siltstone(with clayey bedding)	5.03	4.63	1.09
	Sandstone(some carbonaceous planes)	5.88	7.55	0.78
	Siltstone & Sandstone(some clayey laminae)	5.93	4.11	1.44
C81-23	Claystone	1.37	0.47	2.91
	Siltstone(minimal claystone)	7.48	1.52	4.92
	Siltstone(some clay interbeds)	8.44	2.58	3.27
C81-24	Sandstone & Siltstone(clayey planes)	9.30	2.39	3.89
	Sandstone & Claystone(interbeds)	7.14	1.72	4.15
	Clayey Sandstone & Claystone(interbeds)	5.95	0.98	6.08

POINT LOAD TEST DATA

SOURCE	DESCRIPTION	AVERAGE VALUES, MPA		
		I(AXIAL)	I(DIAMETRAL)	I(ANISOTROPY)
Miscellaneous Roof	Claystone(minor siltstone)	2.39	0.494	4.84
	Sandstone & Siltstone	6.91	2.59	2.67
Roof Caprock	Claystones	2.56	1.27	2.02
Lump Samples	Coal	0.476	0.491	1.03

APPENDIX 3-4

PILLAR DESIGN FOR LONGWALL MINING AT
SMOKY RIVER #4 SEAM
COAL MINING RESEARCH CENTRE

COAL MINING RESEARCH CENTRE
PILLAR DESIGN FOR LONGWALL MINING AT SMOKY RIVER #4 SEAM
BY
K. BARRON AND P.L. WRIGHT
CMRC 82/61-C   OCTOBER 1982

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1. EXECUTIVE SUMMARY

The Coal Mining Research Centre was asked by Smoky River Coals Ltd. to consider three ground control questions pertaining to the design of a longwall mining system layout for the #4 seam in 9G mine.
These questions were:-

1. What is the minimum longwall face length that will ensure satisfactory caving in the gob behind the face ?
2. What are the dimensions of intervening pillars between panels based on design criteria:-
   (a) maintaining a competent pillar for the extraction of both panels ?
   (b) maintaining a competent pillar during the extraction of the first panel but allowing the pillar to yield, without sudden failure, during the extraction of the second panel ?

The analyses presented in this report give the following results and recommendations:-

1. The minimum face length that will ensure satisfactory caving is approximately 430 ft. (130 m). Consequently, with the proposed face length being 500 ft. (150 m) or more, no problems should be experienced in obtaining satisfactory caving behind the face.
2. To ensure a rigid competent pillar during the extraction of the second panel, at a 50% probability of failure, the pillar width would have to be well in excess of 160 ft. (50 m). This would appear to be an unsatisfactory solution due to the excessive cross cut lengths required and the resulting implications with respect to development scheduling. In addition, the rigid nature of this pillar is likely to lead to severe floor heave and consequent maintenance problems along the entry length. This solution is therefore not recommended.
3. It is shown that an intervening pillar, 120 ft. (36 m) wide will:
   (i) behave as a rigid competent pillar during the extraction of the first panel

(ii) yield during the extraction of the second panel when the retreat face abutment load is applied to it.

The yielding of this pillar will tend to minimise, but not eliminate, the effects of floor heave in the immediate vicinity (up to 150 ft. (45 m)) of the retreating face. At the same time, this pillar is judged to be of a sufficient width to avoid sudden failure.

Furthermore, the yielding characteristics of this pillar should tend to minimise the potential for roof break-up that might be experienced with a rigid pillar design. Nonetheless, every precaution should be taken to maintain adequate support and entry height within the zone of influence of the forward abutment pressure.

It is therefore recommended that a 120 ft. (36 m) wide pillar be used in this design layout. Using this pillar width it is suggested that the pillar length should be 300 ft. (90 m).

4. It is recognised that the proposed longwall extraction plan will be the first demonstration of this mining method in these conditions. It is therefore recommended that, when mining commences, the roof and pillar behaviour be monitored by measurement of closure and abutment pressure changes in order to refine the design data.

2. TERMS OF REFERENCE

Figure 1 (contained in back flap at rear of report) shows the plan of the preliminary layout of the longwall retreat mining system for 9G mine at Smoky River Ltd. Grande Cache. The seam is approximately 20 ft. thick and a mining height of 10 ft. is planned. The overburden varies over the panel length from 600 ft. to 1200 ft. Dip varies from 0° to 18° across the proposed faces. The proposed face length is approximately 500 ft.; three such faces are planned for retreat extraction. Figure 2 gives a typical lithologic sections.

The Coal Mining Research Centre were asked by Smoky River Coals Ltd. to advise on the design requirements for ground control for this proposed layout by analysing and making recommendations with respect to:

1. The minimum longwall face length that will ensure satisfactory caving in the gob behind the face.
2. The dimensions of the intervening pillar between the panels based on two design criteria:
   (i) maintaining a competent rigid pillar during the extraction of both panels,
   (ii) maintaining a competent rigid pillar during the extraction of the first panel but allowing the pillar to yield, without sudden failure, during the extraction of the second panel.

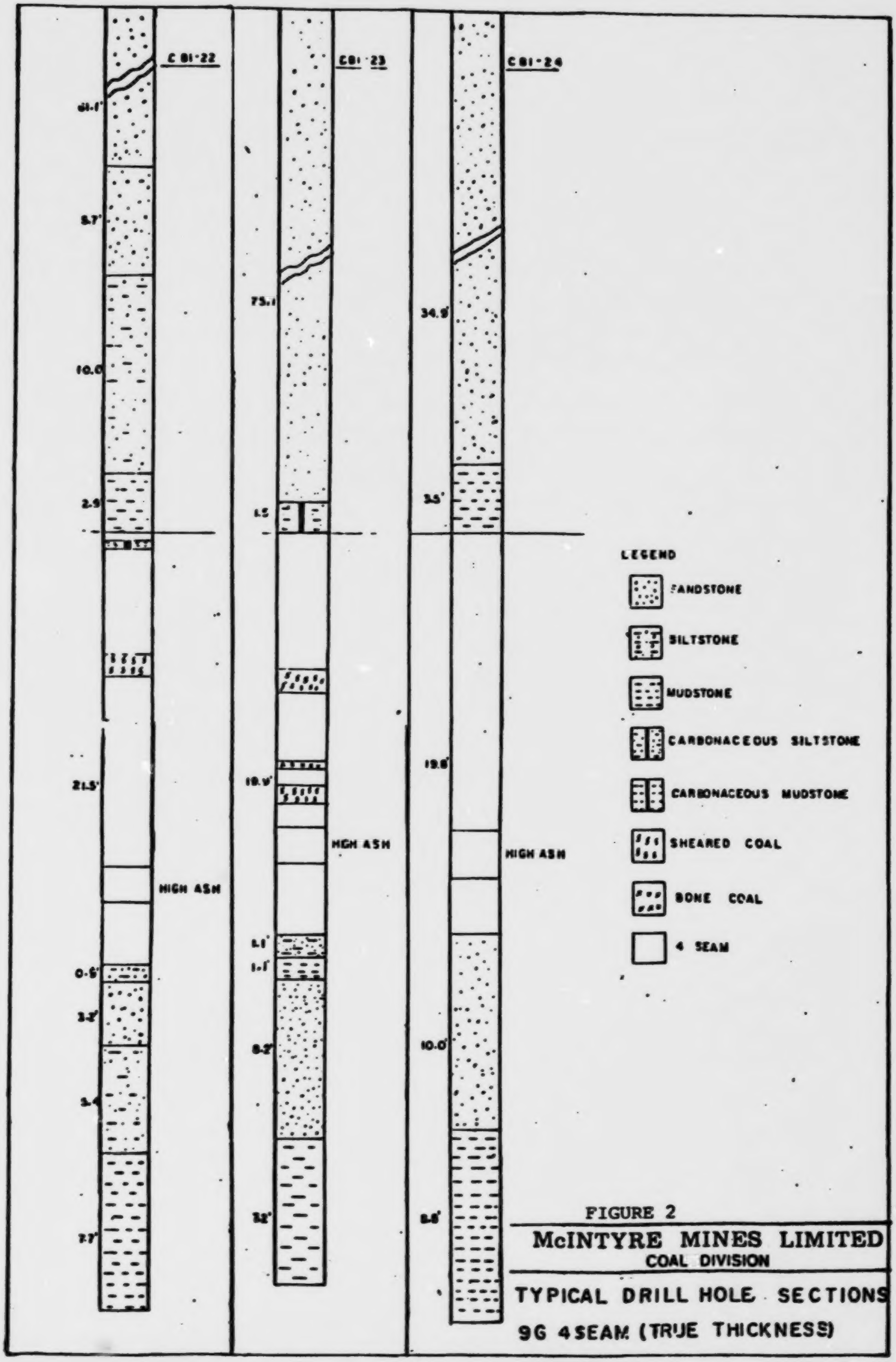

FIGURE 2

McINTYRE MINES LIMITED
COAL DIVISION

TYPICAL DRILL HOLE SECTIONS
9G 4 SEAM (TRUE THICKNESS)

3. DISCUSSION OF THE PRINCIPLES

3.1 Pressure Arch Concept

As soon as an underground opening is formed, the vertical load directly above the opening shifts outwards to both sides of the ribs, leaving a de-stressed or relaxed zone in the roof strata. The shape of the relaxed zone resembles an arch and is thus called the pressure arch. As the opening is widened the arch will lose its integrity at a point called the "maximum pressure arch". Beyond this span the transverse load bearing capacity of the overlying strata ceases, the arch fails and the overburden load will be transferred directly to the immediate roof, the pillars, ribs or to the gob.

3.2 Minimum Span to ensure Caving

At the beginning of a longwall operation the face equipment and supports are contained in a single entry. As mining commences and the face retreats, a pressure arch is formed and an expanse of unsupported roof is left behind the support line; when this span exceeds the maximum pressure arch then gob caving occurs. Just prior to this first caving occurring the maximum load is thrown into the face supports; if the maximum pressure arch is too wide then the loads on the face supports may become excessive leading to major face support control problems.

Clearly if the face length is too short then the lateral confinement will increase the maximum pressure arch span. Consequently to eliminate the effects of lateral confinement, and to ensure that caving takes place at the earliest opportunity, the face length should be equal to or greater than the maximum pressure arch span.

> i.e. to ensure good caving in the gob the face length must be greater than or equal to the maximum pressure arch span.

It is therefore necessary to calculate the maximum pressure arch span. If the overlying strata is massive and competent then beam theory can be used to calculate this span. However this is seldom the case and most certainly not the case for the Smoky River strata where the overlying siltstones and sandstones are fractured. For fractured roof strata it is preferrable to consider the pressure arch as a "linear" or "voussoir" arch. Section 4 gives the concepts and calculations required to determine the maximum pressure arch span using linear arch theory.

3.3 Abutment stresses around a Longwall face

The formation of a longwall face results in the redistribution of strata pressures in the form illustrated by Figure 3. High pressure or abutment stress zones are created in the solid ground surrounding the extracted region. There are two abutment pressure zones created, the face abutment pressure zone and the flank abutment pressure zone. The peak stresses are experienced in the vicinity of the corners where the two abutment pressures interact.

The face abutment pressure exerts its peak pressure 3-10 ft. in advance of the longwall face (1). The magnitude of this peak is governed by the properties of the surrounding rocks; values of four to five times the cover load have often been accepted in British coal measures(1). The distance ahead of the face where the influence of the front abutment is first experienced may be 200 ft. or more.

The flank abutment pressure is time dependent with progressive failure of the sides causing a reduction of the peak flank abutment pressure and a relocation further into the solid.

Clearly, when a second longwall face is mined parallel to the first face with an intervening pillar, then this pillar will be subject to the combined flank abutment pressures of the two faces. Moreover, at the corners of the second face this pressure will be further increased by the interaction with the front abutment pressure of the second face.

It is not possible to calculate exactly the combined effects of these interacting abutments however some approximate calculations can be made for design purposes. These are given in Section 5.

Clearly the intervening pillar must be dimensioned to be of a strength capable of resisting, at least for a period, these peak applied abutment stresses. The calculation of pillar strengths is given in Section 4.

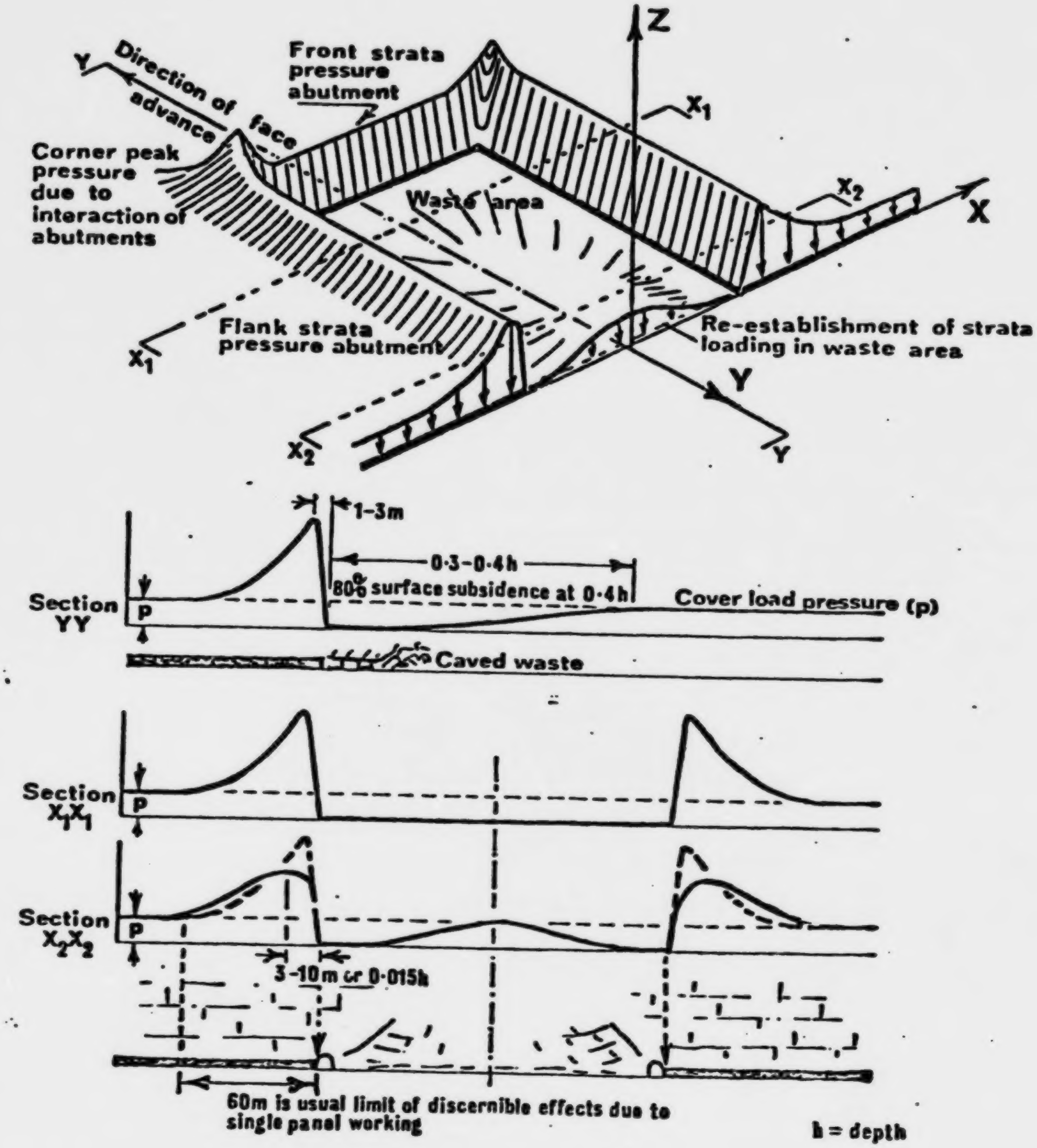
Z
Y
Direction of face advance
Front strata pressure abutment
X1
Corner peak pressure due to interaction of abutments
Waste area
X2
X
Flank strata pressure abutment
Re-establishment of strata loading in waste area
X1
Y
X2
Y
1-3m
0·3-0·4h
80% surface subsidence at 0·4h
Section YY
P
Cover load pressure (p)
Caved waste
Section X1X1
P
Section X2X2
P
3-10m or 0·015h
60m is usual limit of discernible effects due to single panel working
h = depth

FIGURE 3: ABUTMENT STRESSES AROUND A LONGWALL FACE (AFTER WHITTAKER (1))

4. MINIMUM FACE LENGTH FOR SATISFACTORY CAVING

4.1 Stability of a Linear Arch

As discussed in Section 3.1 and 3.2, the minimum face length to ensure satisfactory caving in the gob should be equal to or greater than the maximum pressure arch span. This will be done by considering the pressure arch as a "linear" or voussoir" arch.

4.1.1. Concepts of the roof as a linear arch

Evans (2) demonstrated that fractured roofs may be self supporting in a similar manner to voussoir arches in civil engineering. Consider figure 4 in which an opening is overlain by fractured strata; the two blocks are prevented from falling because they are prevented from rotating about points A and B. The restraining forces preventing rotation are the lateral reactions (shown by arrows); frictional forces at the ends of the blocks resist the shear forces and prevent the blocks from moving vertically downwards.

It is apparent that the fractured rocks are supported entirely by compression and shear resistance and consequently the strength of the linear arch does not depend on the tensile or flexural strength of the rock (unlike beam theory). The stability of the linear arch is a function of its thickness/length ratio, of the compressive strength and the modulus of the rock. The elastic modulus is important since it determines the amount of deformation along the arch line which controls the deflection of the linear arch; this in turn decides the length of the arm, Z, of the resisting moment, as illustrated in Figure 5.

Evans used the compressive strength of the rock and the distance between the centres of pressure to calculate the approximate maximum resisting moment of the linear arch. He then equated the maximum resisting moment to the maximum bending moment of a gravity loaded arch to determine the maximum length of self supporting arch. This is given by:-

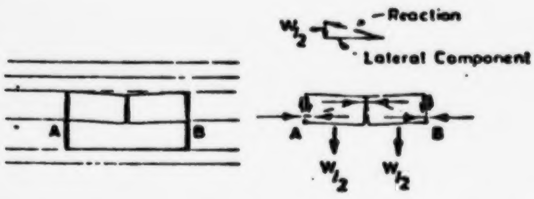

FIGURE 4: ARCHING ACTION IN FRACTURED STRATA
(AFTER EVANS (2))

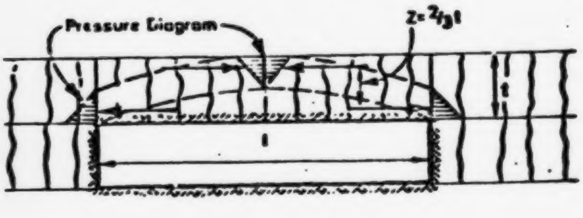

FIGURE 5: ELEMENTS OF THE "LINEAR ARCH"
(AFTER EVANS (2))

$$\ell = (15.49) \left(\frac{Qt}{\gamma}\right)^{1/2} \qquad (1)$$

where ℓ = maximum length of self supporting arch - feet
Q = maximum compressive strength of the rock - psi.
t = thickness, or vertical depth, of the arch - feet
γ = rock density, lbs/cu ft.

This equation only gives a first approximation since it assumes no elastic deformation (i.e. $E = \infty$). It does however give a good approximation for thick linear arches (say t >200 ft.) with relatively high modulus rocks.

In general the elastic deformation of the arch and the abutments must be considered. This effect may be described as follows: the deflection of the beam is due to bending moment and compression along the arch reaction line which has the effect of shortening the arch and thereby reducing the value of Z, the arm of the couple producing the resisting moment. The final deflection is determined by calculating the deflection, determining the increased stress due to deflection, calculating the new deflection, the new increased stress, and repeating these calculations until the stress increase approaches zero.

4.1.2. Calculation of the stability of a linear arch

To determine the maximum span for a self supporting linear arch of thickness t ft. it is necessary to assume a span, ℓ,and calculate the resulting stress and elastic deformation. This operation is repeated until the longest arch is determined in which the compressive stress, f, does not exceed the compressive strength, Q, of the rock. The following are the requisite equations and sequence of steps in this determination:-

(i) Assume a span length ℓ.

(ii) The stress (before deflection) is given by:-

$$f = \frac{\ell^2}{1.54\,t} \qquad (2)$$

(iii) The initial arch height, z, before deflection is given by:-

$$Z = \frac{2}{3} t \tag{3}$$

(iv) The length of the arch line, L, is given by:-

$$L = \ell + \frac{8}{3} \frac{z^2}{\ell} \tag{4}$$

(v) The strain, e, along the arch line is given by:-

$$e = 0.458 \left\{\frac{f}{E}\right\} \tag{5}$$

where E is the rock modulus of elasticity.

(vi) The deformation along the arch line, Δ_1, is given by:

$$\Delta_1 = Le \tag{6}$$

(vii) The deformation, Δ_2, of the abutment is given by:-

$$\Delta_2 = 1.25 \left\{\frac{f\ t}{E}\right\} \tag{7}$$

(Note: this assumes that an average stress of ½ (f/4) exists for a distance 5t into the abutment)

(viii) The total deformation of the arch line, Δ_T, is given by:-

$$\Delta_T = \Delta_1 + \Delta_2 \tag{8}$$

(ix) Hence the revised length, L_1, of the arch line is given by:-

$$L_1 = L - \Delta_T \tag{9}$$

(Note that if $L_1 < \ell$ then the shortened arch line is less than the assumed span, hence collapse must have occurred i.e. the assumed initial span was too long)

(x) The revised height of the arch, Z_1, is given by:-

$$Z_1 = \frac{3}{8}\,\ell\,(L_1 - \ell)^{\frac{1}{2}} \qquad (10)$$

(xi) Now the increase in stress is proportional to the shortening of the resisting arm from Z to Z_1, thus the revised stress, f_1, is given by:

$$f_1 = \frac{Z}{Z_1}\,.\,f \qquad (11)$$

(xii) The total revised deformation (shortening) of the arch, Δ_{T1}, is proportional to (f_1/f) and hence:-

$$\Delta_{T_1} = \Delta_T\,\frac{f_1}{f} \qquad (12)$$

(xiii) Returning now to step (ix), the calculations are repeated to determine L_2, Z_2, f_2 and Δ_{T2}. This calculation process is repeated until the change in stress, δf, from f_{n-1} to f_n is approaching zero.

If $f_n > Q$ then the rock will crush and the arch will fail. The stability of a shorter assumed initial span ℓ should then be examined.

If δf approaches zero and $f_n < Q$ then the arch is stable.

The maximum stable span is the one given when $\delta f \to 0$ and $f_n = Q$.

These equations have been programmed to allow solution by an HP19-C hand calculator. However, before performing these calculations the values of the required input parameters for the #4 seam rocks should be considered.

4.2 Value of the arch thickness, t.

It is not possible to derive an exact value of the arch thickness, t, to be used in these calculations. However a maximum value can be estimated. Clearly, if the arch fails and the void is totally filled due to the expansion of broken rock then this will tend to prevent the arching action progressing upwards. Thus the thickness t of rock which on bulking will completely fill the void can be taken as the maximum value of t.

Now if K is the bulking factor for the fractured rock and if H is the mining height, then to completely fill the void the following equation must be satisfied.

$$H + t = Kt$$

Hence,

$$t = \frac{H}{(K-1)} \qquad (13)$$

e.g. if the bulking factor is 1.25 (25% expansion on fracturing) then $t = 4H$

Using a value of t estimated in this way will given an indication of the maximum arch thickness; this in turn will lead to the calculation of the maximum unsupported arch span. This is particularly useful when wishing to calculate the maximum gob span likely to stand prior to the first caving behind a longwall face.

Table 1, lists bulking factors estimated by various investigators for different types of rock. For this study a value of 1.33 will be taken as representative of the siltstones and sandstones overlying the #4 seam. Thus with a mining height H of 10 ft. then

$$t = \frac{10}{(1.33 - 1)} = 30 \text{ ft.}$$

TABLE 1: BULKING FACTORS FOR DIFFERENT ROCKS

INVESTIGATOR(S)	BULKING FACTOR K	MATERIAL
Kenney (3)	1.53 to 1.64	Coal
	1.7	Shale and Sandstone
	1.45	Marl
Wilson (4)	1.49	Shales ?
Gorrie and Scott (5)	1.33	General rock product
Barry, Nair and Miller (6)	1.12	Sandstones ?
Binns (7)	1.5	Shales ?
Adam, Coeillet and Jacob (8)	> 1.2	Shales ?

4.3 Value of the maximum compressive strength Q, and the modulus E.

Table 2 gives values of the uniaxial compressive strength of the #4 seam rocks determined in a geotechnical study of this property (9). It will be noted that the sandstone is the strongest element of the roof strata and has a mean strength of 11400 psi with a maximum value of 16900 psi. Consequently for these calculations this maximum value will be used: this will reflect the effects of the strongest possible roof strata and this leads to the determination of the maximum possible unsupported arch span.

Table 3 gives the static elastic modulus determined from this same study (9) for these rocks; the modulus of these small specimens ranges from 4.8 x 10^6 to 8.1 x 10^6 psi. However it is not reasonable to take the small specimen modulus as being respresentative of the modulus of the linear arch since the linear arch is fractured and must have a lower modulus. For calculation purposes a value of 1 x 10^6 psi will be used, although this is still likely to be a high estimate and again result in the determination of the maximum possible unsupported arch span.

Table 4 gives the bulk densities for these rocks (9). A value of 165 lbs/cu.ft. will be used for these arch calculations.

4.4 Maximum pressure arch for Smoky River

Using the input data derived above, the stability for a number of assumed spans, ℓft, was analysed by the technique given in Section 4.1.2. The following results were obtained:-

Bulk density = 165 lbs/cu ft.
Maximum compressive strength = 16900 psi
Arch thickness t = 30 ft.
Modulus, E = 1 x 10^6 psi.
Assumed span ℓ = 500 ft. - unstable
400 ft. - stable
450 ft. - unstable

TABLE 2

UNIAXIAL COMPRESSIVE STRENGTH

TEST RESULTS

Rock Type	Test Hole	Depth Feet	Depth (m)	Height Width Ratio	Compressive Strength psi	Compressive Strength (MPa)	Average psi	Average (MPa)
Sandstone (fresh)	75-94	598	(182.3)	1.5	8260	(57.0)		
	104	51	(15.5)	1.0	16914	(116.6)		
	104	96	(29.3)	1.0	11336	(78.2)		
	73-45	945	(28.8)	1.0	11762	(81.1)	11387	(78.5)
	73-45	159	(48.5)	1.0	16305	(112.4)		
	75-94	592	(180.4)	2.0	12730[1]	(87.8)		
	14	15.5	(4.7)	1.0	10333[1]	(71.3)		
	14	17	(5.2)	1.0	3459[1]	(23.8)		
Siltstone (fresh)	101	91	(27.7)	1.0	15631	(107.8)		
	103	104	(31.7)	1.0	8365	(57.7)	8865	(61.1)
	103	41	(12.5)	1.0	2598[1]	(17.9)		
Mudstone (fresh)	105	103	(31.4)	1.0	1141	(7.9)	1141	(7.9)
Coal (weak,wet)	12 12	15.5 16.5	(4.7) (5.0)	1.0	5.5	(0.0)	5.5	(0.0)
Coal (banded, strong)	70-23	not known		1.0	1137	(7.8)	1137	(7.8)

[1] Sample is slightly weathered and altered.

TABLE 3

Static Elastic Properties

Sample No.	Rock Type	Modulus of Elasticity E_s psi (MPa)	Poisson's Ratio μ_s
592	Sandstone (medium)	8.1×10^6 (55.9×10^3)	0.14
298	Sandstone (fine)	7.4×10^6 (51.0×10^3)	0.12
598	Sandstone (fine)	7.8×10^6 (53.8×10^3)	0.16
288	Siltstone	4.8×10^6 (33.1×10^3)	0.16
298[1]	Sandstone (fine)	6.6×10^6 (45.5×10^3), C_p = 150 psi	
		7.5×10^6 (51.7×10^3), C_p = 750 psi	

[1] Triaxial test results at confining pressures (C_p) of 150 psi and 750 psi.

TABLE 4

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BULK DENSITY TEST RESULTS

Rock Type	Test Hole	Depth		Bulk Density		Average Bulk Density	
		Feet	(m)	pcf	t/m^3	pcf	t/m^3
Sandstone	75-94	592	(180.4)	168.2	(2.69)	167.7	(2.69)
	75-94	600	(182.9)	165.8	(2.66)		
	75-94	602	(183.5)	168.0	(2.69)		
	75-94	307	(93.6)	166.1	(2.66)		
	75-94	298	(90.8)	169.4	(2.71)		
	75-94	301	(91.7)	163.0	(2.61)		
	104	51	(15.5)	168.7	(2.70)		
	104	96	(29.3)	166.8	(2.67)		
	73-45	94.5	(28.8)	168.1	(2.69)		
	73-45	159	(48.5)	169.0	(2.71)		
	14	15.5	(4.7)	170.0	(2.72)		
	14	17	(5.2)	169.3	(2.71)		
Siltstone	75-94	285	(86.9)	156.9	(2.51)	160.4	(2.57)
	75-94	288	(87.8)	161.7	(2.59)		
	75-94	587	(178.9)	162.7	(2.61)		
Mudstone	75-94	308	(93.9)	150.2	(2.41)	158.0	(2.53)
	75-94	310	(94.5)	162.0	(2.59)		
	75-94	605-609	(184.4-185.6)	161.9	(2.59)		
Coal (weak, wet)	12	15.5	(4.7)	83.6	(1.34)	83.7	(1.34)
	12	16.5	(5.0)	83.8	(1.34)		
Coal, (strong, banded)	70-23	not known	not known	93.5	(1.50)	93.5	(1.50)

425 ft. - stable
435 ft. - unstable
430 ft. - stable

Hence the maximum pressure arch for Smoky River lies between 430 -435 ft. It should be noted that natural conditions will seldom be sufficiently favorable as to permit the existence of such a long span as is calculated by this method. (Factors tending to reduce the length of the self supporting span would include diagonal fractures, clay seams, time dependent rock deformation etc.).

Consequently, as outlined in Section 3.2, to ensure good caving in the gob the face length should be greater than or equal to the maximum pressure arch span.

4.5 Conclusion

The minimum face length to ensure satisfactory gob caving is 430 ft. (In practice it is unlikely that a span as long as this will be self supporting). Hence, with the proposed face length being 500 ft. or more, no problems should be experienced with obtaining satisfactory gob caving.

5. PILLAR STRENGTHS

In order to dimension the pillar between the two longwall panels it is necessary to determine the variation of pillar strength with the minimum pillar width, (B), and the variation of the stresses applied to the pillar during the longwall retreat. In this section the variation of pillar strength with minimum pillar lateral dimension will be considered.

5.1 Pillar strength formulae

Table 5 summarises five different pillar strength formulae developed by various authors, namely Holland and Gaddy (10), Salamon (11), Bieniawski (12), Hustrulid (13) (as adapted for #4 seam coal by Barron (14)), and Barron (15).

The first four of these formulae require the input of a "cube coal strength", either k for a 1 inch cube or K for a 1 ft. cube depending on whether the pillar dimensions in the formulae are expressed in inches or in feet. Now Barron (14) estimated the value of k for the No. 4 seam coal in No. 2 mine as 6400 ± 1775. Using Gaddy's (17) relationship between cube strengths, $\sigma c = \frac{k}{D^{0.5}}$ where D is the cube dimension in inches, and k = 6400 then K = 1847. These input values have been inserted in the formulae listed in Table 1 to give the appropriate mean pillar strength equation for No.4 seam coal for each of these formulae. These equations are listed in the final column of Table 5.

The fifth set of equations, Barron (15), represents a new, and as yet not fully proven, derivation of pillar strength formulae which include both brittle fracture and yield criteria. These equations are considerably more complex and require the knowledge of additional coal parameters for the No. 4 seam coal. These parameters have not been measured for the No. 4 seam coal; however Barron (16) obtained a good fit between the fracture zones measured in three pillars in No. 4 seam (No. 2 mine) by air injection tests and the fracture zones predicted by this analytical technique when using the following parameters:-

P_B = 2 psi.
$Q_B = 29^{\circ}$
μB = 0.4
and A = 534 psi.

TABLE 5

SUMMARY OF PILLAR STRENGTH FORMULAE

AUTHOR [REFERENCE]	MEAN PILLAR STRENGTH FORMULA	UNITS/DIMENSIONS	FORMULA WITH NO.4 SEAM COAL CONSTANTS
		$\bar{\sigma}_p$ = MEAN PILLAR STRENGTH – P.S.I.	h = COAL CONSTANT FOR 1 INCH CUBE : K = THAT FOR 1 FT CUBE BARRON [14] OBTAINED h = 6400 ± 1775 FOR NO.4 SEAM COAL USING THIS VALUE AND GADDY'S RELATIONSHIP [17] FOR CUBE STRENGTHS : $\sigma_c = \frac{h}{D^{0.5}}$ GIVES K = 1847 (WHERE D = CUBE DIMENSION IN INCHES.
HOLLAND/GADDY [10]	$\bar{\sigma}_p = h\left(\frac{B^{0.5}}{H}\right)$	B = PILLAR WIDTH – INCHES H = PILLAR HEIGHT – INCHES	$\bar{\sigma}_p = 6400\left(\frac{B^{0.5}}{H}\right)$ P.S.I.
SALAMON [11]	$\bar{\sigma}_p = K\left(\frac{B^{0.46}}{H^{0.66}}\right)$	B = PILLAR WIDTH – FEET H = PILLAR HEIGHT – FEET	$\bar{\sigma}_p = 1847\left(\frac{B^{0.46}}{H^{0.66}}\right)$ P.S.I.
BIENIAWSKI [12]	$\frac{\bar{\sigma}_p}{\sigma_c} = \left(0.645 + 0.355\left[\frac{B}{H}\right]\right)$ WHERE : WHEN B < 5 FT $\sigma_c = K\left(\frac{B^{0.16}}{H^{0.55}}\right)$ WHEN B > 5 FT $\sigma_c = K\left(\frac{5^{0.16}}{5^{0.55}}\right) = 0.534\ K$	B = PILLAR WIDTH – FEET H = PILLAR HEIGHT – FEET σ_c = CUBE STRENGTH FOR CUBE OF SIDE = B FT. IN PSI	FOR B > 5 FT THEN $\sigma_c = 0.534[1347] = 956$ PSI HENCE :- $\bar{\sigma}_p = 636 + 350\left(\frac{B}{H}\right)$ P.S.I.
HUSTRULID – BARRON [13 & 14]	$\frac{\bar{\sigma}_p}{\sigma_c} = \left(0.778 + 0.222\left[\frac{B}{H}\right]\right)$ WHERE : WHEN H < 36 INS $\sigma_c = \frac{h}{H^{0.5}}$ WHEN H > 36 INS $\sigma_c = \frac{h}{36^{0.5}} = 0.167\ h$	B = PILLAR WIDTH – INCHES H = PILLAR HEIGHT – INCHES σ_c = CUBE STRENGTH FOR CUBE OF SIDE = H INS. IN P.S.I.	FOR H > 36 INS. THEN $\sigma_c = 0.167[6400] = 1068$ PSI HENCE :- $\bar{\sigma}_p = 830 + 237\left(\frac{B}{H}\right)$ P.S.I.
BARRON [15]	$\bar{\sigma}_{P(mm)} = \frac{\gamma}{\gamma_{mm}} \cdot 2A$ $\boxed{1}$ OR IF $d_t > b$ $\bar{\sigma}_p = \frac{P_B(1-\sin\phi_B)}{2b\sin\phi_B}\left\{\frac{h}{\tan\phi_B}\left[e^{\tan\phi_B\left(\frac{1+\sin\phi_B}{1-\sin\phi_B}\right)\left(\frac{b}{h}\right)} - 1\right] - b\right\}$ $\boxed{2}$ OR IF $d_t < b$ $\bar{\sigma}_p = \frac{P_B(1-\sin\phi_B)}{2b\sin\phi_B}\left\{\frac{h}{\tan\phi_B}\left[e^{\tan\phi_B\left(\frac{1+\sin\phi_B}{1-\sin\phi_B}\right)\left(\frac{d_t}{h}\right)} - 1\right] - d_t\right\} + \left\{\frac{d_t - b}{b}\right\}P_{IT}$ $\boxed{3}$ WHERE : $d_t = \left(\frac{1-\sin\phi_B}{1+\sin\phi_B}\right)\frac{h}{\tan\phi_B}\ \log_e\left\{\frac{2P_{IT}\sin\phi_B}{P_B(1+\sin\phi_B)} + \left(\frac{1-\sin\phi_B}{1+\sin\phi_B}\right)\right\}$ AND $P_{IT} = \frac{A}{2} + \frac{A}{2\mu_B^2}\left\{1 + \left(1+\mu_B^2\right)^{0.5}\right\} + \left\{\frac{A^2}{2} + \frac{A^2}{2\mu_B^2}\left[1 + \left(1+\mu_B^2\right)^{0.5}\right]\right\}^{0.5}$ AND $A = 2\sigma_c$ WHERE $\sigma_c = \frac{h}{36^{0.5}} = 0.167\ h$ FOR H > 36 INCHES		FOR NOMENCLATURE AND UNITS SEE TEXT BARRON [16] FITTED PILLAR AIR INJECTION TEST RESULTS FROM NO.4 SEAM USING THE FOLLOWING VALUES FOR THE PARAMATERS :- $P_B = 2$ PSI. $\phi_B = 29°$ $\mu_B = 0.4$ $A = \frac{0.167[6400]}{2} = 534$ PSI.

(Appendix I shows the comparison so obtained for the three pillars in No. 4 seam, No. 2 mine). These input parameters will therefore be used to calculate the pillar strengths in this approach. The nomenclature for Barron's formulae listed in Table 1 is as follows:-

σ_p (min) = minimum pillar strength - psi.
Y= mean pillar stress concentration factor
Y_{mm} = maximum pillar stress concentration factor at the mid height on the pillar boundary.
P_B = uniaxial compressive strength of broken coal - psi.
ϕ_B = angle of internal friction of broken coal - degrees
b = semi-pillar width - feet or inches.
h - semi-pillar height - feet or inches
σ_p = pillar strength - psi.
d_T = depth of the transition point from brittle to psuedo- ductile behaviour - feet or inches.
P_{IT} = vertical stress at the transition point - psi.
μ_B = coefficient of sliding friction of coal on coal.
A = Mohr constant for solid coal (when Mohr envelope is expressed in form $\tau^2 = A\sigma + (\frac{A^2}{4})$

5.2 Variation of pillar strength with minimum pillar width B.

The mean pillar strength has been calculated using each of the formulae given in Table 5, and the appropriate coal parameters for No. 4 seam given above for a range of pillar widths from 50 to 200 ft. In all cases the pillar height H was taken as equal to the proposed face height of 10 ft.

Table 6 tabulates the results and Figure 6 gives a graph of the results. It will be noted that the Bieniawski formula yields relatively high pillar strengths whilst the Holland/Gaddy formula gives consistently low pillar strengths. The other three formulae give relatively similar results, particularly in the range 80 ft. to 150 ft.

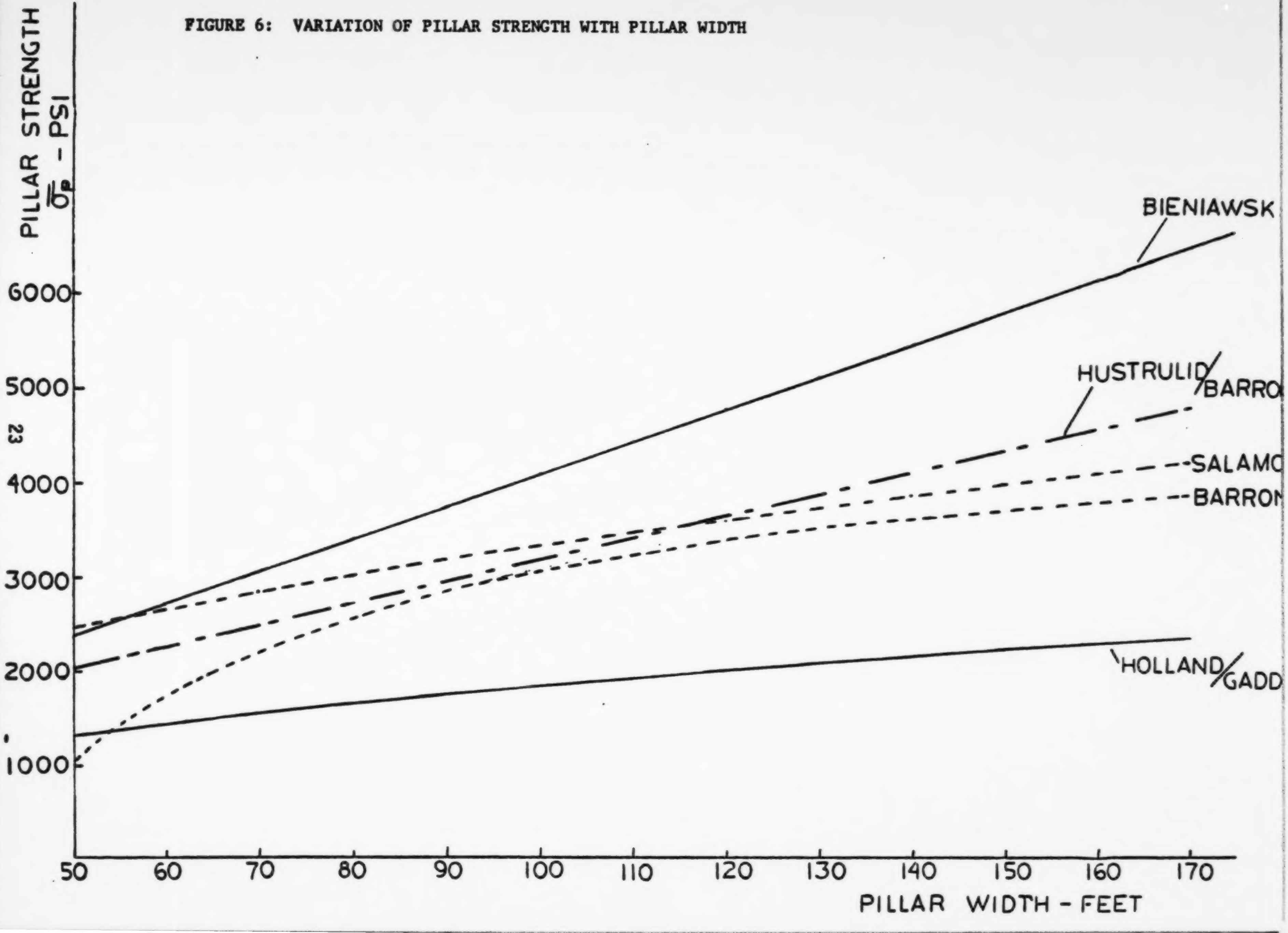
It should be noted that, using Barron's formula, the mode of failure will be one of pseudo-ductile failure (yield) for all pillar widths in excess of 46.4 ft., with brittle failure below this width.

TABLE 6: CALCULATED MEAN PILLAR STRENGTHS FOR 10 FT. HIGH PILLARS - # 4 SEAM

σ_p psi

FORMULA PILLAR WIDTH - B FT.	HOLLAND/ GADDY	SALAMON	BIENIAWSKI	HUSTRULID/ BARRON	BARRON
50	1306	2443	2386	2015	1004
60	1430	2657	2736	2252	1694
70	1545	2852	3086	2489	2187
80	1652	3033	3436	2726	2557
90	1752	3201	3786	2963	2844
100	1847	3360	4136	3200	3074
110	1937	3511	4486	3437	3262
120	2023	3655	4836	3674	3419
130	2105	3792	5186	3911	3552
140	2185	3923	5536	4148	3666
150	2262	4050	5886	4385	3764
160	2336	4172	6236	4622	3851
170	2408	4290	6586	4859	3927
180	2478	4404	6936	5096	3994
190	2545	4515	7286	5333	4055
200	2612	4623	7636	5570	4110

FIGURE 6: VARIATION OF PILLAR STRENGTH WITH PILLAR WIDTH



6. PILLAR STRESSES

6.1 Estimate of the Flank Abutment Stresses during the Mining of the First Panel

Consider Figure 7, let L be the face length, B the pillar width, D the pillar length, W the entry width and C the cross cut width. Let the seam be at a depth Z below the surface. It is assumed that there is a constant angle of shear (angle of draw), $\alpha°$ to the horizontal; extending out over the gob. The strata below this angle is assumed to cave and the weight is taken by the gob. The weight of the strata above this angle is taken by the abutments. It is also assumed that the depth of abutment over which this load is shed is that defined by drawing the same angle on the other side of the vertical. i.e. the weight of strata in the shaded area A_1 in Figure 7 is distributed over an abutment depth Z Cot α. It is seen from the two cases in Figure 1 that (a) when $L/2 >$ Cot α, the area A_1 is constant regardless of the face length whereas (b) when $L/2 < Z$ Cot α the area A_1 varies with face width. The average abutment stress can be calculated for these two cases in the manner given below:-

Case 1: $\frac{L}{2} \geq Z \text{ Cot } \alpha$

Consider the shaded area A_1:

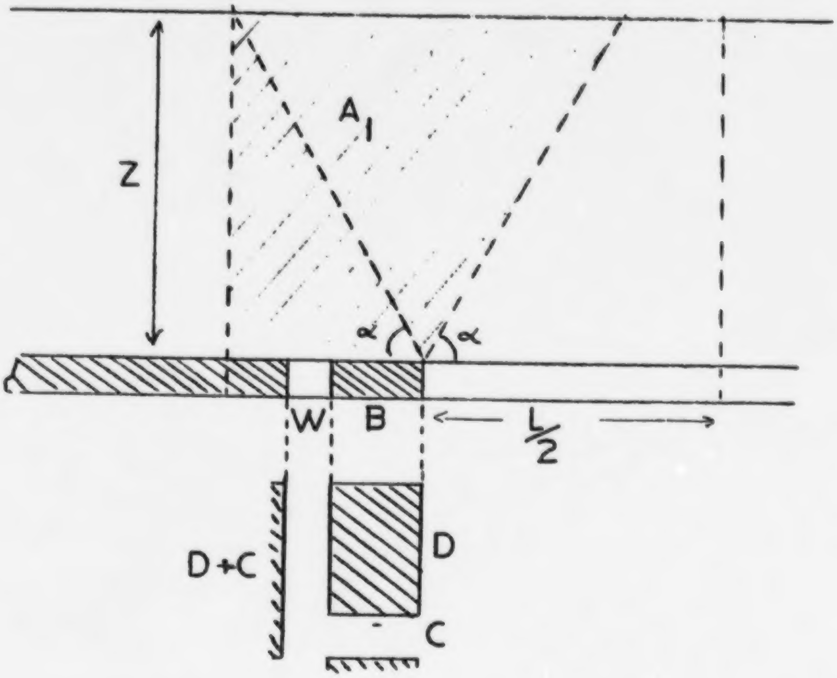
$$A_1 = \frac{3}{2} Z^2 \text{ Cot } \alpha \qquad (14)$$

The total weight, T_W, of rock supported over pillar plus cross cut length (D+C) is:-

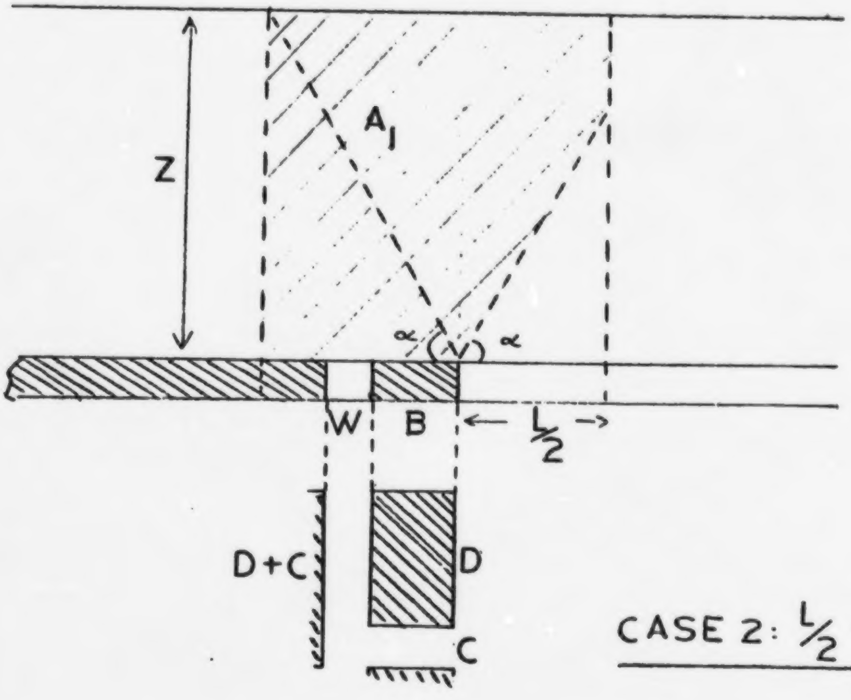
$$T_W = A_1 \text{ (D+C) } \gamma \qquad (15)$$

where γ is the bulk density of the overlying strata.

If Z Cot α > (B+W) then this weight is shared between the pillar and the solid seam and the mean pillar stress (flank abutment stress) is given by $\bar{\sigma}_p$:-



CASE 1: $L/2 > Z \cot\alpha$



CASE 2: $L/2 < Z \cot\alpha$

FIGURE 7: FLANK ABUTMENT AFTER MINING FIRST PANEL

$$\bar{\sigma}_p = \frac{T_w}{(D+C)\ \{Z \text{ Cot } \alpha - B - W\} + BD} \qquad (16)$$

or if Z Cot $\alpha < (B+W)$ then this weight is supported by the pillar only and the mean pillar stress is given by:-

$$\bar{\sigma}_p = \frac{T_w}{B.D} \qquad (17)$$

and the mean abutment stress concentration factor $\frac{\bar{\sigma}_p}{Z\gamma}$ (18)

Case 2: $\frac{L}{2} \leq Z \text{ Cot } \alpha$

Consider the shaded area A_1:

$$A_1 = Z^2 \text{ Cot } \alpha + \frac{LZ}{2} - \frac{L^2}{8} \tan \alpha \qquad (19)$$

The total weight of rock supported over the pillar plus cross cut length (D+C) is Tw:-

$$T_w = A_1\ (D+C)\ \gamma \qquad (20)$$

If Z Cot $\alpha > (B+W)$ then this weight is shared between the pillar and the solid seam and the mean pillar stress (mean flank abutment stress) is given by $\bar{\sigma}_p$:

$$\bar{\sigma}_p = \frac{T_w}{(D+C)\ \{Z \text{ Cot } \alpha - B - W\} + BD} \qquad (21)$$

If Z Cot $\alpha < (B+W)$ then this weight is supported by the pillar only, hence the mean pillar stress is given by:-

$$\bar{\sigma}_p = \frac{T_w}{BD} \qquad (22)$$

and the mean abutment stress concentration factor = $\frac{\bar{\sigma}_p}{Z\gamma}$ (23)

Both of these cases have been programmed for solution using an HP19C calculator.

6.2 Estimate of the Flank Abutment Stress on the Intervening Pillar During Mining of the Second Panel

A similar approach can be taken in estimating the stresses applied to the pillar between the two panels, as illustrated in Figure 8. Again there are two cases (a) when $L/2 > Z \cot \alpha$ and (b) $L/2 < Z \cot \alpha$.

Case 1: $\frac{L}{2} > Z \cot \alpha$

Consider the shaded area A_1:-

$$A_1 = Z^2 \cot \alpha + ZB \qquad (24)$$

The total weight of rock supported over the pillar plus cross cut length (D+C) is T_W:-

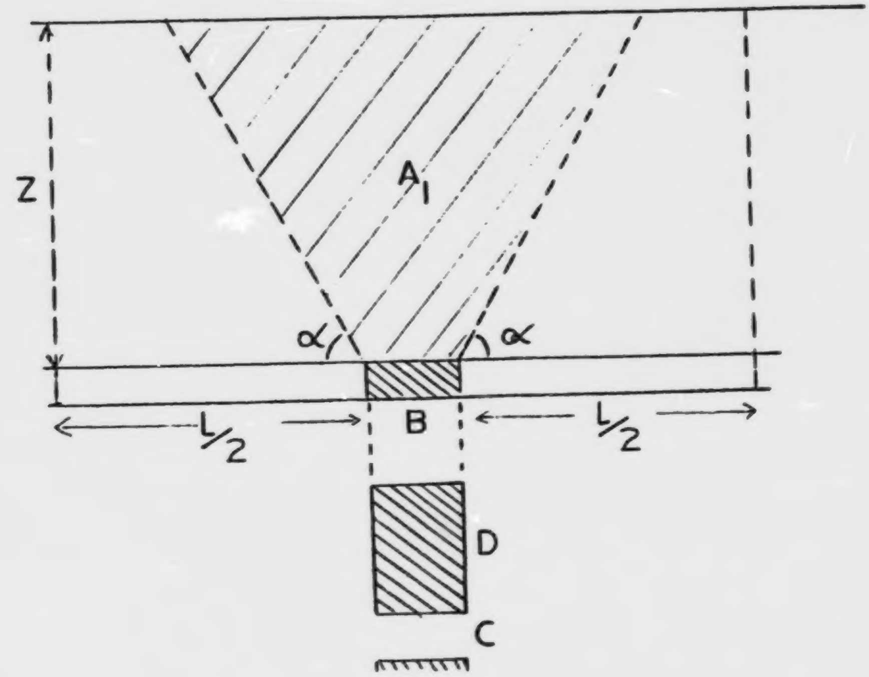
$$T_W = A_1 \, (D+C) \, \gamma \qquad (25)$$

Hence the mean pillar stress, $\bar{\sigma}_p$ is given by:

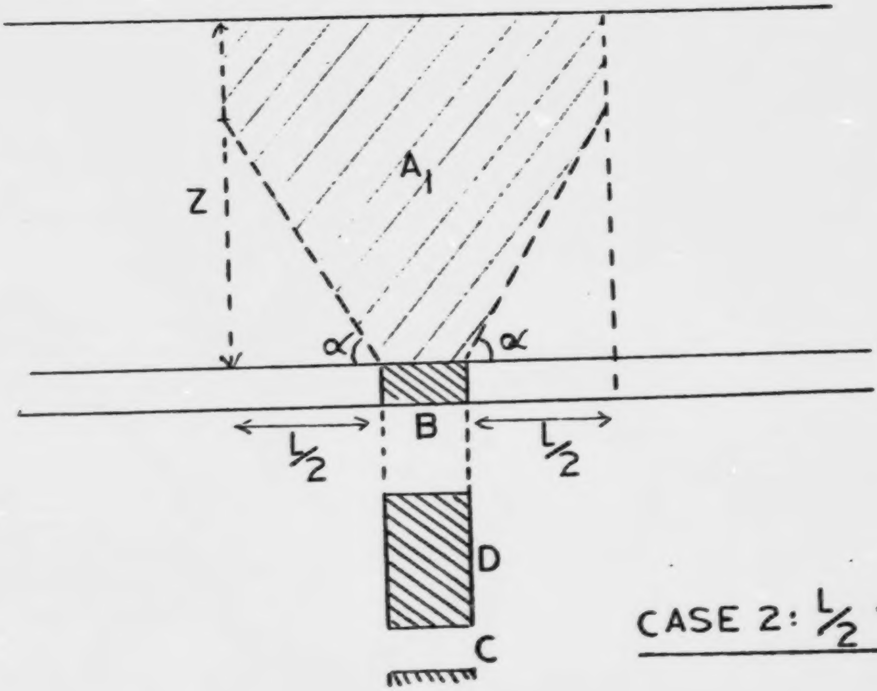
$$\bar{\sigma}_p = \frac{T_W}{B.D} \qquad (26)$$

and the mean pillar stress concentration factor is = $\frac{\bar{\sigma}_p}{Z\gamma}$ (27)

Case 2: $\frac{L}{2} < Z \cot \alpha$



CASE 1: $L/2 > Z \cot \alpha$



CASE 2: $L/2 < Z \cot \alpha$

FIGURE 8: PILLAR LOADS AFTER MINING TWO PANELS

Consider the shaded area A_1:-

$$A_1 = LZ + BZ - \frac{L^2}{4} \tan \alpha \qquad (28)$$

The total weight of the rock supported over the pillar plus cross cut length (D+C) is T_W:-

$$T_w = A_1 \ (D+C) \ \gamma \qquad (29)$$

Hence the mean pillar stress $\bar{\sigma}_p = \frac{T_w}{B.D}$ (30)

and the mean pillar stress concentration factor is $= \frac{\bar{\sigma}_p}{Z\gamma}$ (31)

These equations have also been programmed for solution using an HP19C calculator.

6.3 Input Parameter for the 9G, #4 Seam Layout, and Stresses Determined

The following input parameters were asumed for the 9G, 4 seam layout under consideration.

1. Entry width W = cross cut width C = 20 ft.
2. Pillar width B, was varied.
3. The pillar width B plus the pillar length D was kept constant at 420 ft.. This distance was chosen since it represents the standard "move up" length for continuous miner conveyor positions.
4. The face length L = 500 ft.
5. The bulk density γ = 165 lbs/cu ft.
6. The angle of shear $\alpha = 60^{\circ}$. Note this is a relatively conservative estimate based on generally accepted National Coal Board Data for U.K. conditions. However, the use of this 60° angle will give higher abutment stresses than using a steeper angle; thus the design will be conservative.
7. The overburden over these panels varies from 600 ft. to 1200 ft.. The full flank abutment will not be applied until caving takes place, i.e. until the

face has retreated say 430 ft.. Consequently the overburden will have reduced from the maximum 1200 ft. to approximately 1000 ft. by the time the full flank abutment is applied. These calculations will therefore be based on an overburden depth of 1000 ft. Clearly, for overburden less than this amount the abutment stress will be correspondingly less. However, for design purposes this conservative value of 1000 ft. will be used.

The mean flank abutment stresses for the single panel and the two panel cases have been calculated using the above input parameters and the equations derived above, for a range of pillar widths from 50 ft. to 200 ft. Table 7 gives these results which are also shown in Figure 9.

The following points should be noted:

1. During extraction of the first panel, the flank pillar stress concentration factor remains almost constant with pillar width at approximately 1.4. With a constant 1000 ft. overburden this corresponds to a stress of 1600 psi.
2. During extraction of the second panel, the flank intervening pillar stress concentration factor reduces rapidly with increasing pillar size from approximately 9.3 at a 50 ft. pillar to 3.2 with a 200 ft. pillar. With a constant 1000 ft. overburden this corresponds to stresses of 10670 psi and 3670 psi respectively.

It must be borne in mind that the stresses calculated above estimate only the flank stresses; they do not take into account the additional stress concentration at the face corners due to the interaction of the front and flank abutments. This interaction may increase these peak stresses by perhaps 1.3 - 1.5 times the flank abutment stresses.

The only way to attempt to estimate the peaks caused by the abutment interactions is to perform a complete stress analysis model of this situation. Such a model study is underway but the results are not available at the time of writing. An addendum to this report will be submitted when these model study results are available.

FIGURE 9: MEAN PILLAR STRESS ON FLANK ABUTMENT DURING FIRST AND SECOND PANEL EXTRACTION

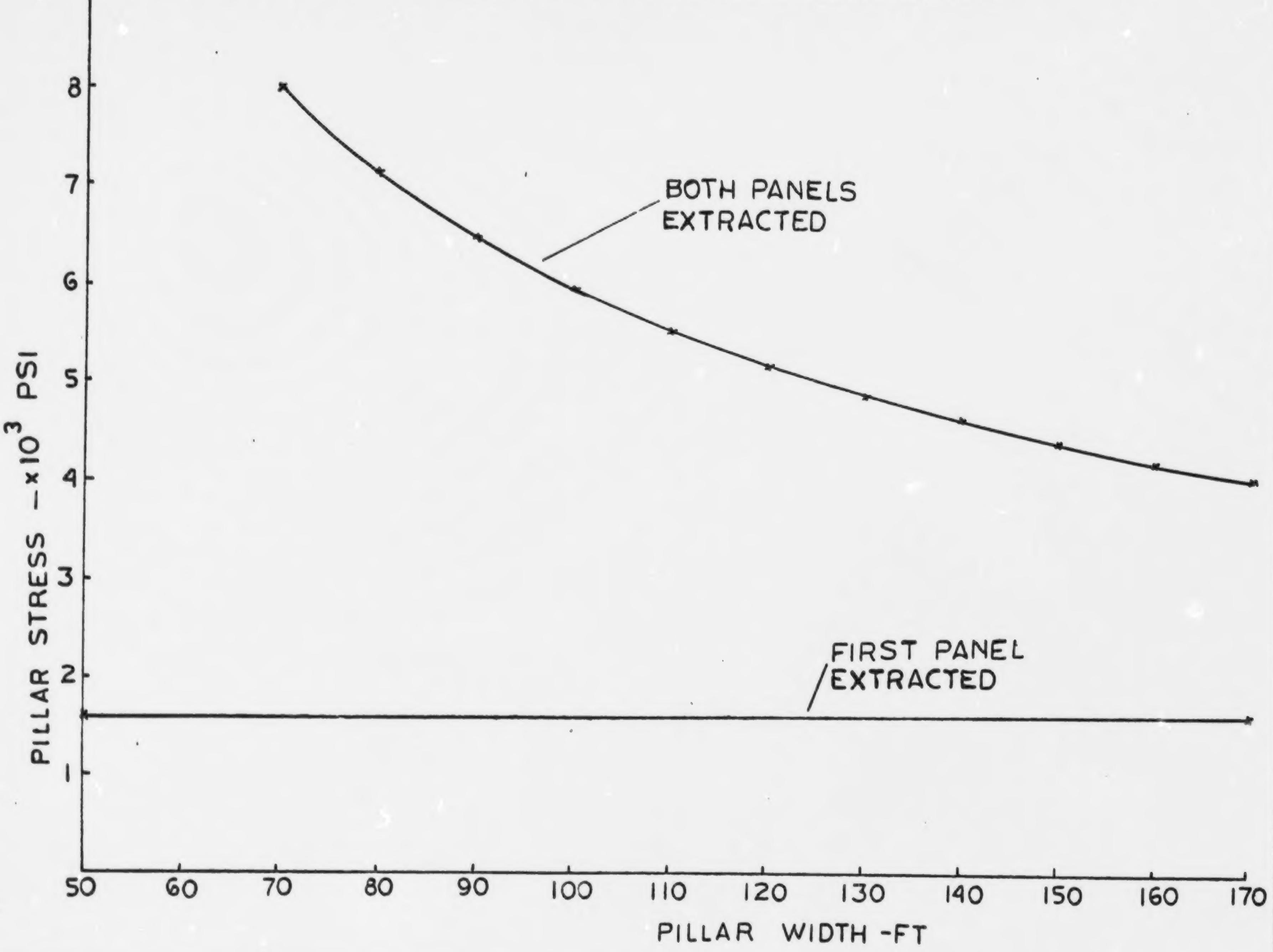


Table 7: Mean Pillar Stresses on Flank Abutment During First and Second Panel Extraction

Pillar Width B-ft.	Pillar Length D-ft.	First Panel Stress Concentration Factor	σ_p psi	First Plus Second Panel Stress Concentration Factor	$\bar{\sigma}_p$ psi
50	370	1.394	1597	9.31	10670
60	360	1.395	1598	7.95	9106
70	350	1.396	1600	6.97	7990
80	340	1.398	1602	6.24	7154
90	330	1.400	1604	5.68	6505
100	320	1.402	1606	5.22	5987
110	310	1.404	1608	4.86	5563
120	300	1.406	1611	4.55	5212
130	290	1.408	1614	4.29	4915
140	280	1.411	1617	4.07	4662
150	270	1.413	1620	3.88	4445
160	260	1.416	1623	3.71	4255
170	250	14.19	1626	3.57	4089
180	240	1.422	1630	3.44	3943
190	230	1.426	1634	3.32	3813
200	220	1.427	1635	3.20	3673

$D+B = 420$ ft.

$C = W = 20$ ft.

$Z = 1000$ ft.

$\gamma = 165$ lbs/cu ft.

$L = 500$ ft.

$\alpha = 60^{\circ}$

7. PILLAR DESIGN DISCUSSION

7.1 "Best" Strength Estimate

The variation of pillar strength with pillar width was calculated in Section 5 by five different formulae, as shown in Figure 6. It was noted that the extremes of these estimated strengths were given by Bieniawski's (high estimate) and Holland and Gaddy's (low estimate) formulae. The other three approaches gave very similar results. Consequently the extreme estimates will be ignored and the mean of the results of the other three calculations will be taken as the "best" estimate of pillar strength. The variation of this "best" strength estimate with pillar width is shown as curve #5 in Figure 10.

Also plotted on this figure are the flank stresses calculated during the mining of the first panel (curve #1), a graph representing 1.3 times this value (curve #2) which gives the effect of the front and flank abutment interactions assuming an additional stress concentration factor of 1.3. Curve #3 on this figure gives the flank pillar stresses for the mining of the second panel and curve #4 is 1.3 times this value, again assuming a 1.3 stress concentration factor due to abutment interaction.

This figure 10 will form the basis of the following design discussion.

7.2 Mining of the First Panel

It is seen from Figure 10 that at no stage (in the range of pillars sizes considered) does the flank abutment pressure during mining the first panel (curve #1) intersect the "best" strength curve. i.e. the safety factor is always greater than 1.

If it is assumed that interaction between the flank and forward abutments gives an additional stress concentration factor of 1.3, then curve #2 results. This curve intersects the "best" strength curve at a pillar width of approximately 55 ft. (i.e. safety factor = 1 at this width).

Consequently if a pillar width of in excess of 55 ft. is chosen, it can be expected to be stable (not to fail) at any stage during mining of the first panel.

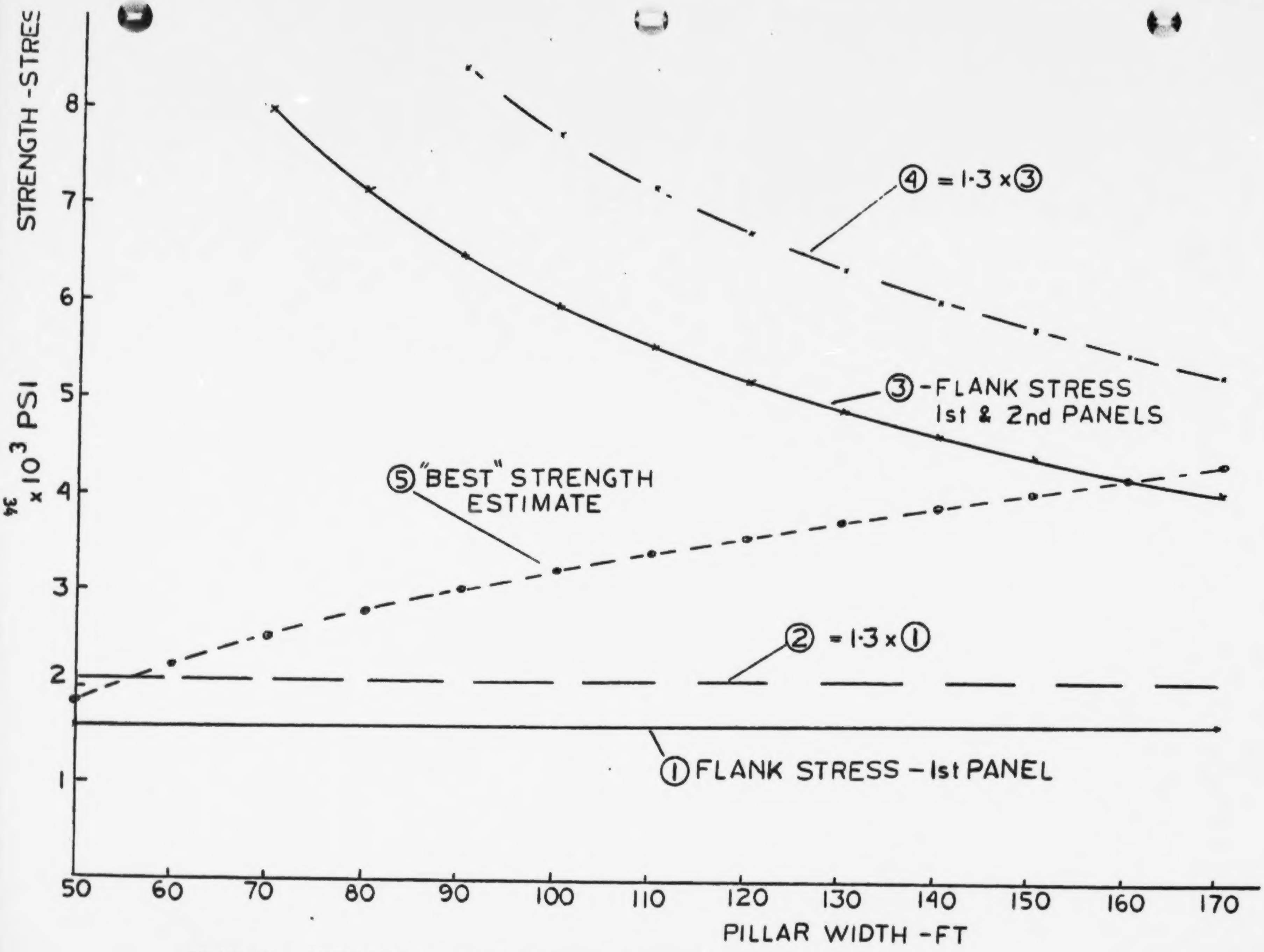


FIGURE 10: COMPARISON OF PILLAR STRESSES AND STRENGTHS

7.3 Mining of the Second Panel

Consider now the flank stresses on the pillar when the second panel is mined (curve #3). This intersects the "best" strength estimate curve at a pillar width of approximately 160 ft. Even without allowing for the abutment interaction increase of 1.3 (curve #4) this appears to be an excessive pillar width for practical purposes.

i.e. To ensure that the pillar does not fail during mining the second panel would require a pillar width well in excess of 160 ft. From a practical point of view having to cut cross cuts of 160 ft. or more in a two entry system would cause significant scheduling difficulties for continuous miners and would reduce productivity. In addition, bearing in mind the relatively weak nature of the underlying coal floor, the rigid nature of such a wide pillar would be likely to lead to severe floor heave and consequent maintenance problems along the entry length.

A choice of pillar dimension to prevent failure during the mining of the second panel is therefore both difficult and impractical and consequently is not recommended.

A yield pillar design is therefore the only satisfactory compromise. In such a design, first and foremost, the pillar must not fail suddenly by brittle failure but should yield slowly in a controlled manner. It was noted in Section 5 that only Barron's (15) analytical approach to pillar analysis addresses the actual mode of failure (the other authors do not address the mode of failure). According to Barron's analysis pillars less than 46 ft. will fail by brittle failure whereas larger pillars will fail by a pseudo-ductile yielding mechanism.

In accepting a yield pillar design the other factor that must be accepted is that the pillars and entries will deform during yielding and additional support will have to be placed in the entries to control this deformation. Clearly it would be desirable to limit the distance ahead of the retreating face over which such additional support and maintenance will be required; whilst at the same time not having so wide a pillar as to give excessice cross cut lengths.

Consider Figure 11 which shows a sketch of the panels and pillar during extraction of the second panel. At some distance ahead of the face, at

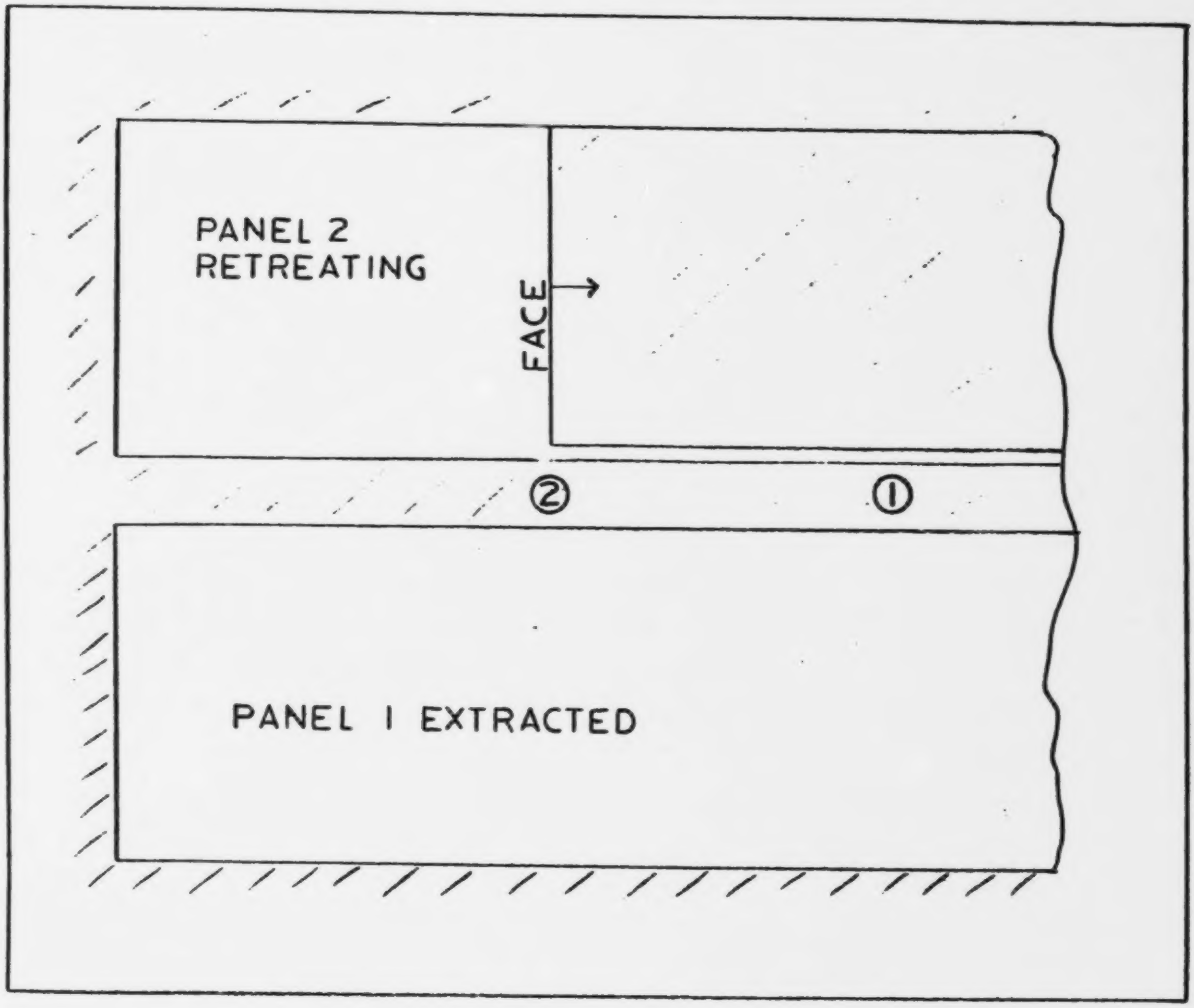


FIGURE 11: EXTRACTION OF THE SECOND PANEL

position 1 on Figure 11, the pillar is subject only to the flank abutment stresses due to mining of the first panel. At position 2 on this figure, adjacent to the face, the pillar will be subject to the peak stresses caused by the interaction of the face abutment and the flank abutments from both pillars. Table 8 gives the calculated stresses for both these situations (assuming the 1.3 times interactive concentration at position 2), the strengths and the resulting safety factor for the range of pillar sizes from 60 - 160 ft. The safety factor at point 1 will be called the "initial safety factor" and that a point 2 will be called the "Minimum" or final safety factor.

Let us assume that the retreat of the second face first influences the pillar at a distance of 500 ft. ahead of the face. i.e. the "initial" safety factor applies at this point (which is > 1 for all pillar sizes considered). Adjacent to the face, at position 2, the minimum safety factor will apply (which is < 1 for all the pillar sizes considered). Thus the safety factor changes from the "initial" to the minimum over this 500 ft. and will pass through the safety factor of 1.0 somewhere in this 500 ft. interval. Once the safety factor reduces below unity the pillar will start to yield and the entries deform. In Figure 12A the safety factors for the two positions for each of the pillar sizes have been plotted against distance ahead of the face and have been joined by straight lines; thus Figure 12B can be plotted indicating the distance ahead of the face where the safety factor equals unity, for the range of pillar sizes. i.e. for any pillar size the length of entry ahead of the face which is likely to be affected by the pillar yielding is approximated in Figure 12B.

(in practice, of course, the curves in Figure 12A will not be linear, the slope will be flatter further away from the face and steeper closer to the face. Thus Figure 12B will overestimate the position ahead of the face where yielding will take place. Likewise the influence of the face may not extend as far as 500 ft. from the face, in which case Figure 12B will again overestimate the pillar yield distance ahead of the face).

On this basis therefore we can select a pillar size which is compatable with reasonably practical cross cut lengths whilst at the same time limiting the distance over which entry deformation problems have to be tackled by additional support and maintenance.

TABLE 8: SAFETY FACTOR VARIATION WITH PILLAR SIZE

Pillar Width	Best Strength	Flank - 1st Panel		Peak - 2nd Panel	
FT.	PSI.	STRESS	S.F.	STRESS	S.F.
60	2201	1598	1.37	11837	.186
70	2509	1600	1.57	10387	.241
80	2772	1602	1.73	9300	.298
90	3002	1604	1.87	8456	.355
100	3211	1606	2.00	7783	.413
110	3403	1608	2.12	7231	.470
120	3582	1611	2.24	6775	.529
130	3811	1614	2.38	6389	.596
140	3912	1617	2.45	6060	.645
150	4066	1620	2.54	5778	.704
160	4215	1623	2.63	5531	.762

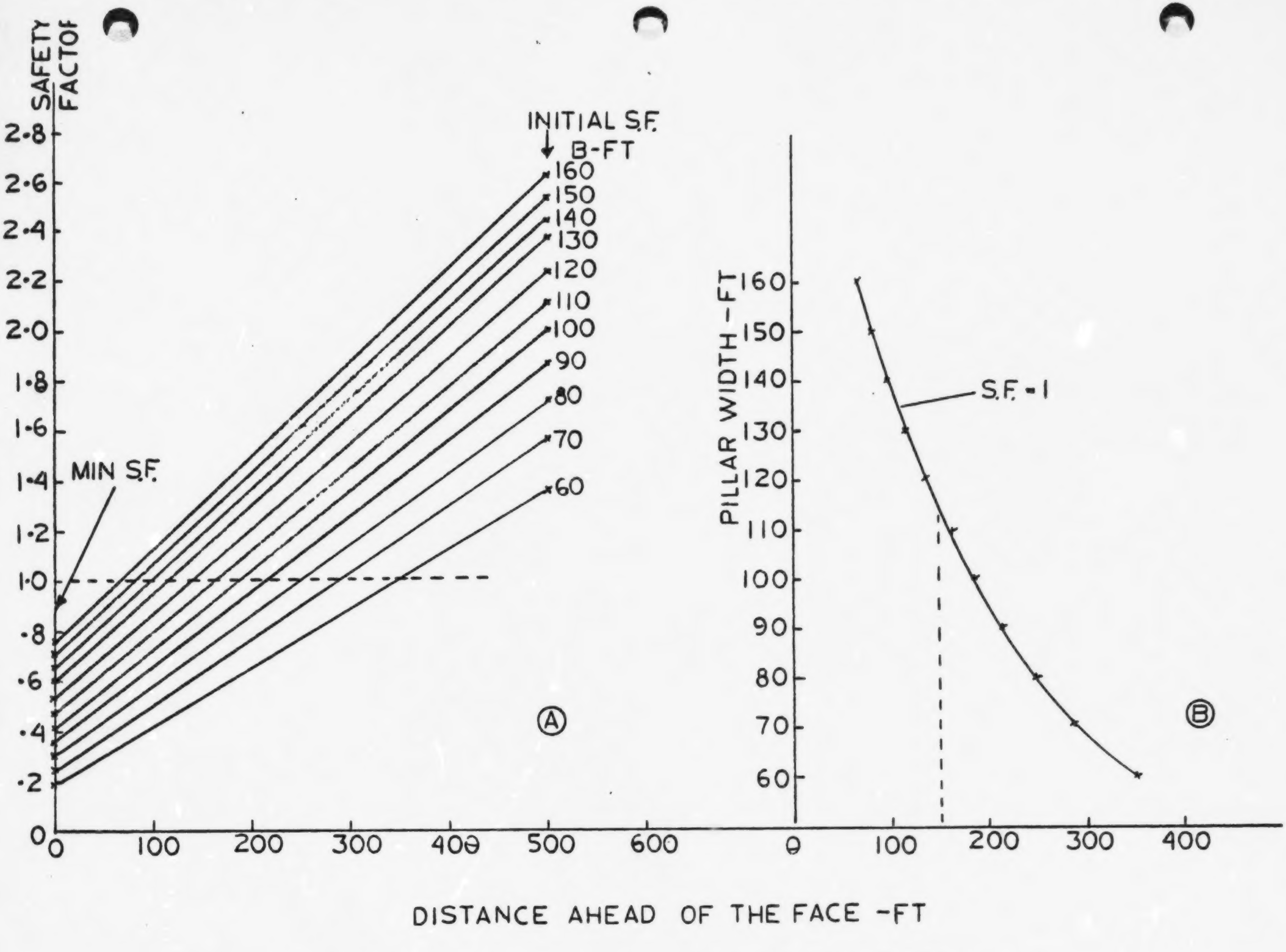


FIGURE 12: SAFETY FACTOR VARIATION WITH PILLAR SIZE AND DISTANCE AHEAD OF THE FACE

On this basis a 120 ft. pillar size is recommended. 120 ft. cross cuts are reasonably practical from a development point of view and, as seem from Figure 12B, the entry length affected by yielding should not exceed 150 ft.

In addition the yielding of the pillar when the major stresses are encountered will tend to minimize, but not eliminate, the effects of floor heave in the entry ahead of the face. At the same time the pillar will not fail suddenly.

Furthermore the yielding characteristics of this pillar should tend to reduce the potential for roof breakup that might be experienced with a rigid pillar design.

Nonetheless, every precaution must be taken to maintain support and entry height, by artificial support and good maintenance, within the zone of influence where the forward abutment pressure causes pillar yielding, which with this 120 ft. pillar, may extend up to 150 ft. ahead of the face.

Using this 120 ft. pillar design width and keeping the sum of the pillar width and length equal to 420 ft. for convenience of conveyor move up, determines the pillar length between cross cuts as 300 ft.

8. RECOMMENDATIONS

1. It is recommended that a minimum face length of 430 ft. be considered in order to ensure satisfactory gob caving.
2. It is recommended that a 120 ft. wide and 300 ft. long pillar be used between the two panels. During extraction of the first panel this pillar will not fail. i.e. it will remain a rigid relatively competent pillar during this extraction phase.

   During the extraction of the second panel this pillar will yield when subject to the influence of the forward abutment of the second retreating face. It is estimated that, with a 120 ft. wide pillar, yielding of the pillar should not commence more than 150 ft. ahead of the face.
3. Every precaution should be taken within this 150 ft. distance ahead of the face to ensure that the entries are adequately supported and that adequate entry height is maintained.
4. It is recognized that the proposed longwall extraction plan will be the first demonstration of this mining method and these concepts in these conditions. It is therefore recommended that, when mining commences, the roof and pillar behaviour be monitored by measurement of closure and abutment pressure changes in order to refine this design data.

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APPENDIX I

Barron (15) has developed an analytical technique for determining the strengths, the fracture zones and stress distributions across pillars. This analytical technique is now being applied to study a number of case histories of pillars taken from the literature. Amongst these case histories under study were three pillars in #2 mine, #4 seam at Grande Cache; field air injection tests were carried out in these pillars to indicate the fracture zones and the central core of the pillar. The air injection index distribution across the pillar has the same shape as the stress distribution curve. Analytical input parameters were chosen for the #4 seam coal in order to compare the analytically derived stress distribution across the pillar with the air injection results. Table A1 gives the stress distribution derived from the analytical technique using the following input parameters:-

P_B = 2 psi
ϕ_B 29^o
μ_B =0.4
A = 534 psi

Figure A1 compares these stress distributions with the air injection test results. It is seen that the depth of the peripheral fracture zone, the position of the abutment stress peaks and the area of an "undisturbed" elastic core all compare favorably.

These input parameters have therefore been applied in the text to calculate the pillar strengths for #4 seam coal pillars.

P R - A

```
CALCULATION FOR RECTANGULAR OPENING

VIRGIN STRESS=1540.0        EXTRACTION RATIO=0.307
LENGTH OF SPAN= 610.0       SEMI-WIDTH OF PILLAR= 30.0
SEMI-HEIGHT OF PILLAR= 5.0  OVERBURDEN DEPTH= 1400
DISTANCE "X"=325.00           SEMI-ROOM WIDTH= 10.0
RATIO "SH/SV"=0.330         NUMBER OF PILLARS IN SPAN=13
COMPRESSIVE STRENGTH OF BROKEN COAL=  2.
MOHR CONSTANT "A"=  500.00
ANGLE OF INTERNAL FRICTION OF BROKEN COAL=30.
POISSONS RATIO FOR SURROUNDING ROCK=0.30
POISSONS RATIO FOR PILLAR ROCK=0.30
COEFFICIENT OF SLIDING FRICTION FOR BROKEN COAL=0.40
YOUNGS MODULAS FOR SURROUNDING ROCK= 5000000
YOUNGS MODULAS FOR PILLAR ROCK= 1000000
STRESS REDISTRIBUTION COEFFICIENT=0.50
TOTAL NUMBER OF SLICES=   304
SLICE INTERVAL IN PRINTOUT=   4


THE RESULTS ARE

MEAN STRESS CONCENTRATION=1.307
MEAN PILLAR STRESS:LMS=2105.
MAXIMUM STRESS CONCENTRATION=    2.18
THE PILLAR IS STABLE
THE FAILURE MODE WILL BE ULTIMATE YIELD
THE PILLAR STRENGTH =   2292.71
THE SAFETY FACTOR   =       1.09
THE CRITICAL PILLAR DIMENSION TO AVOID
       A) CATASTROPHIC YIELD IS  22.07
       B) ULTIMATE YIELD IS  35.37
             K1=              1.0050
             K2=             -0.2259
             K3=              0.0073

  SLICE    (BO-O)  TAUS/F    TAU      STATUS      PONE   PTHREE ACCUM STRESS
  -----    ------  ------    ---      ------      ----   ------ ------------
  1( 0.3)  30.00    433.0  1449.8     FAILED        2.1     0.1     0.0
  8( 2.0)  30.01    433.9  1562.5     FAILED        4.5     1.0   618.2
 12( 3.0)  30.01    434.7  1592.6     FAILED        6.6     1.7   839.9
 16( 4.0)  30.02    435.8  1623.3     FAILED        9.5     2.8  1035.7
 20( 5.0)  30.03    437.3  1656.2     FAILED       13.5     4.3  1217.9
 24( 6.0)  30.05    439.4  1692.7     FAILED       19.0     6.3  1392.9
 28( 7.0)  30.07    442.3  1733.0     FAILED       26.5     8.9  1564.6
 32( 8.0)  30.09    446.0  1777.3     FAILED       36.9    12.6  1735.8
 36( 9.0)  30.13    451.2  1825.4     FAILED       51.1    17.8  1906.3
 40(10.0)  30.17    458.2  1877.5     FAILED       70.8    24.9  2083.7
 44(11.0)  30.23    467.4  1932.3     FAILED       97.8    34.6  2262.9
 48(12.0)  30.31    479.7  1992.5     FAILED      135.0    48.1  2446.8
 52(13.0)  30.42    495.8  2054.8     FAILED      186.3    66.6  2635.4
 56(14.0)  30.56    516.7  2119.9     FAILED      256.8    92.1  2826.6
 60(15.0)  30.74    543.3  2185.1     FAILED      353.9   127.1  3025.3
 64(16.0)  30.98    576.9  2249.8     FAILED      487.5   175.4  3223.3
 68(17.0)  31.29    618.6  2310.5     FAILED      671.4   241.8  3418.7
 72(18.0)  31.70    669.9  2362.1     FAILED .    924.6   333.2  3605.4
 76(19.0)  32.23    732.3  2396.8     FAILED     1273.1   459.0  3773.5
 80(20.0)  32.93    807.4  2403.4     FAILED     1752.7   632.3  3907.0
 84(21.0)  33.83    897.3  2360.1     FAILED     2412.9   870.7  3984.8
 88(22.0)  35.02   1004.1  2240.2     FAILED     3321.7  1198.8  3969.4
 92(23.0)  36.60   1130.6  1996.6     FAILED     4572.5  1650.5  3807.7
 96(24.0)  38.76   1280.0  1556.3     FAILED     6294.3  2272.3  3418.6
100(25.0)  39.55   1298.1  1353.8    YIELDED     4617.4  2272.3  3173.7
104(26.0)  39.55   1298.1  1250.9  NOT FAILED    4836.3  2272.3  2994.3
108(27.0)  39.55   1298.1  1173.8  NOT FAILED    4662.9  2272.3  2834.2
112(28.0)  39.55   1298.1  1104.4  NOT FAILED    4521.6  2272.3  2705.5
116(29.0)  39.55   1298.1  1045.0  NOT FAILED    4400.7  2272.3  2596.4
120(30.0)  39.55   1298.1   993.0  NOT FAILED    4294.7  2272.3  2501.7
124(31.0)  39.55   1298.1   946.7  NOT FAILED    4200.4  2272.3  2418.0
128(32.0)  39.55   1298.1   905.0  NOT FAILED    4115.5  2272.3  2343.2
132(33.0)  39.55   1298.1   867.1  NOT FAILED    4038.3  2272.3  2275.6
136(34.0)  39.55   1298.1   832.4  NOT FAILED    3967.6  2272.3  2214.1
140(35.0)  39.55   1298.1   800.4  NOT FAILED    3902.5  2272.3  2157.7
144(36.0)  39.55   1298.1   770.8  NOT FAILED    3842.2  2272.3  2105.7
148(37.0)  39.55   1298.1   743.3  NOT FAILED    3786.1  2272.3  2057.6
152(38.0)  39.55   1298.1   717.6  NOT FAILED    3733.8  2272.3  2012.8

BRITTLE/DUCTILE TRANSITION POINT:TAUF=   1298.1
FRACTURE ZONE ENDED ON  97TH SLICE AND EXTENDED 24.3 FT INTO THE PILLAR.
YEILD ZONE ENDED ON 103TH SLICE AND EXTENDED 25.8 FT INTO THE PILLAR.
DEPTH OF FRACTURE ZONE JUST AT FAILURE 23.04 .
```

PILLAR -

```
CALCULATION FOR RECTANGULAR OPENING

VIRGIN STRESS=1540.0        EXTRACTION RATIO=0.318
LENGTH OF SPAN= 440.0       SEMI-WIDTH OF PILLAR= 44.5
SEMI-HEIGHT OF PILLAR= 5.0  OVERBURDEN DEPTH= 1400
DISTANCE "X"=270.00           SEMI-ROOM WIDTH= 10.0
RATIO "SH/SV"=0.330         NUMBER OF PILLARS IN SPAN= 3
COMPRESSIVE STRENGTH OF BROKEN COAL=  2.
MOHR CONSTANT "A"=  500.00
ANGLE OF INTERNAL FRICTION OF BROKEN COAL=30.
POISSONS RATIO FOR SURROUNDING ROCK=0.30
POISSONS RATIO FOR PILLAR ROCK=0.30
COEFFICIENT OF SLIDING FRICTION FOR BROKEN COAL=0.40
YOUNGS MODULAS FOR SURROUNDING ROCK= 5000000
YOUNGS MODULAS FOR PILLAR ROCK= 1000000
STRESS REDISTRIBUTION COEFFICIENT=0.50
TOTAL NUMBER OF SLICES=   356
SLICE INTERVAL IN PRINTOUT=   4


THE RESULTS ARE

MEAN STRESS CONCENTRATION=1.383
MEAN PILLAR STRESS:LMS=2120.
MAXIMUM STRESS CONCENTRATION=    2.18
THE PILLAR IS STABLE
THE FAILURE MODE WILL BE ULTIMATE YIELD
THE PILLAR STRENGTH =   2661.40
THE SAFETY FACTOR   =       1.25
THE CRITICAL PILLAR DIMENSION TO AVOID
       A) CATASTROPHIC YIELD IS  22.92
       B) ULTIMATE YIELD IS  35.69
             K1=              1.4312
             K2=              0.0708
             K3=              0.0569

  SLICE    (BO-O)  TAUS/F    TAU      STATUS      PONE   PTHREE ACCUM STRESS
  -----    ------  ------    ---      ------      ----   ------ ------------
  1( 0.3)  30.00    433.0  1447.6     FAILED        2.1     0.1     0.0
  8( 2.0)  30.01    433.9  1572.2     FAILED        4.5     1.0   568.9
 12( 3.0)  30.01    434.7  1605.1     FAILED        6.6     1.7   773.3
 16( 4.0)  30.02    435.8  1635.0     FAILED        9.5     2.8   953.1
 20( 5.0)  30.03    437.3  1664.9     FAILED       13.5     4.3  1119.0
 24( 6.0)  30.05    439.4  1696.3     FAILED       19.0     6.3  1276.5
 28( 7.0)  30.07    442.3  1729.4     FAILED       26.5     8.9  1429.0
 32( 8.0)  30.09    446.0  1764.7     FAILED       36.9    12.6  1578.9
 36( 9.0)  30.13    451.2  1802.4     FAILED       51.1    17.8  1727.6
 40(10.0)  30.18    458.2  1842.2     FAILED       70.8    24.9  1876.3
 44(11.0)  30.24    467.4  1884.1     FAILED       97.8    34.6  2025.5
 48(12.0)  30.33    479.7  1927.7     FAILED      135.0    48.1  2175.6
 52(13.0)  30.44    495.8  1972.4     FAILED      186.3    66.6  2326.5
 56(14.0)  30.58    516.7  2017.3     FAILED      256.8    92.1  2477.4
 60(15.0)  30.78    543.3  2061.3     FAILED      353.9   127.1  2627.0
 64(16.0)  31.04    576.9  2101.8     FAILED      487.5   175.4  2772.9
 68(17.0)  31.39    618.6  2136.0     FAILED      671.4   241.8  2911.2
 72(18.0)  31.84    669.9  2158.8     FAILED      924.6   333.2  3036.1
 76(19.0)  32.43    732.3  2162.7     FAILED     1273.1   459.0  3138.7
 80(20.0)  33.22    807.4  2136.1     FAILED     1752.7   632.3  3206.2
 84(21.0)  34.24    897.3  2061.2     FAILED     2412.9   870.7  3219.5
 88(22.0)  35.58   1004.1  1910.9     FAILED     3321.7  1198.8  3150.6
 92(23.0)  37.35   1130.6  1644.3     FAILED     4572.5  1650.5  2959.0
 96(24.0)  39.70   1280.0  1200.8  NOT FAILED    4541.0  2097.7  2585.5
100(25.0)  39.91   1280.0  1129.0  NOT FAILED    4391.9  2097.7  2454.1
104(26.0)  40.13   1280.0  1060.4  NOT FAILED    4249.7  2097.7  2328.8
108(27.0)  40.31   1280.0  1001.8  NOT FAILED    4128.5  2097.7  2223.5
112(28.0)  40.48   1280.0   950.8  NOT FAILED    4023.4  2097.7  2133.5
116(29.0)  40.63   1280.0   905.8  NOT FAILED    3930.6  2097.7  2055.0
120(30.0)  40.77   1280.0   865.4  NOT FAILED    3847.7  2097.7  1985.7
124(31.0)  40.90   1280.0   828.9  NOT FAILED    3772.7  2097.7  1923.7
128(32.0)  41.02   1280.0   795.6  NOT FAILED    3704.4  2097.7  1867.7
132(33.0)  41.14   1280.0   764.9  NOT FAILED    3641.8  2097.7  1816.8
136(34.0)  41.24   1280.0   736.6  NOT FAILED    3583.7  2097.7  1770.1
140(35.0)  41.34   1280.0   710.3  NOT FAILED    3529.9  2097.7  1727.1
144(36.0)  41.44   1280.0   685.7  NOT FAILED    3479.8  2097.7  1687.3
148(37.0)  41.53   1280.0   662.7  NOT FAILED    3433.0  2097.7  1650.3
152(38.0)  41.63   1280.0   641.1  NOT FAILED    3388.9  2097.7  1615.8
156(39.0)  41.71   1280.0   620.7  NOT FAILED    3347.5  2097.7  1583.4
160(40.0)  41.79   1280.0   601.5  NOT FAILED    3308.4  2097.7  1553.0
164(41.0)  41.88   1280.0   583.3  NOT FAILED    3271.4  2097.7  1524.4
168(42.0)  41.94   1280.0   566.0  NOT FAILED    3236.2  2097.7  1497.3
172(43.0)  42.01   1280.0   549.6  NOT FAILED    3202.9  2097.7  1471.7
176(44.0)  42.08   1280.0   533.8  NOT FAILED    3171.1  2097.7  1447.5

BRITTLE/DUCTILE TRANSITION POINT:TAUF=   1298.1
FRACTURE ZONE ENDED ON  95TH SLICE AND EXTENDED 24.0 FT INTO THE PILLAR
DEPTH OF FRACTURE ZONE JUST AT FAILURE 23.04
```

PILLAR -C

```
CALCULATION FOR RECTANGULAR OPENING

VIRGIN STRESS=1540.0        EXTRACTION RATIO=0.294
LENGTH OF SPAN= 319.0       SEMI-WIDTH OF PILLAR= 42.5
SEMI-HEIGHT OF PILLAR= 5.0  OVERBURDEN DEPTH= 1400
DISTANCE "X"=149.00           SEMI-ROOM WIDTH= 10.0
RATIO "SH/SV"=0.330         NUMBER OF PILLARS IN SPAN= 3
COMPRESSIVE STRENGTH OF BROKEN COAL=  1.
MOHR CONSTANT "A"=  500.00
ANGLE OF INTERNAL FRICTION OF BROKEN COAL=30.
POISSONS RATIO FOR SURROUNDING ROCK=0.30
POISSONS RATIO FOR PILLAR ROCK=0.30
COEFFICIENT OF SLIDING FRICTION FOR BROKEN COAL=0.40
YOUNGS MODULAS FOR SURROUNDING ROCK= 5000000
YOUNGS MODULAS FOR PILLAR ROCK= 1000000
STRESS REDISTRIBUTION COEFFICIENT=0.50
TOTAL NUMBER OF SLICES=   348
SLICE INTERVAL IN PRINTOUT=   4


THE RESULTS ARE

MEAN STRESS CONCENTRATION=1.288
MEAN PILLAR STRESS:LMS=1983.
MAXIMUM STRESS CONCENTRATION=    2.18
THE PILLAR IS STABLE
THE FAILURE MODE WILL BE ULTIMATE YIELD
THE PILLAR STRENGTH =   2372.04
THE SAFETY FACTOR   =       1.20
THE CRITICAL PILLAR DIMENSION TO AVOID
       A) CATASTROPHIC YIELD IS  25.16
       B) ULTIMATE YIELD IS  37.53
             K1=              0.7514
             K2=             -0.4063
             K3=              0.9217

  SLICE    (BO-O)  TAUS/F    TAU      STATUS      PONE   PTHREE ACCUM STRESS
  -----    ------  ------    ---      ------      ----   ------ ------------
  1( 0.3)  30.00    433.0  1441.2     FAILED        1.1     0.0     0.0
  8( 2.0)  30.00    433.5  1498.1     FAILED        2.3     0.5   559.1
 12( 3.0)  30.01    433.9  1508.3     FAILED        3.3     0.9   751.0
 16( 4.0)  30.01    434.4  1522.4     FAILED        4.8     1.4   917.4
 20( 5.0)  30.02    435.3  1541.1     FAILED        6.7     2.1  1070.0
 24( 6.0)  30.03    436.2  1564.4     FAILED        9.5     3.1  1214.8
 28( 7.0)  30.04    437.7  1592.0     FAILED       13.2     4.4  1355.3
 32( 8.0)  30.05    439.6  1623.5     FAILED       18.4     6.3  1494.2
 36( 9.0)  30.07    442.3  1658.8     FAILED       25.6     8.9  1633.0
 40(10.0)  30.10    445.9  1697.5     FAILED       35.4    12.4  1773.0
 44(11.0)  30.13    450.8  1739.6     FAILED       48.9    17.3  1915.1
 48(12.0)  30.18    457.4  1784.7     FAILED       67.5    24.0  2060.2
 52(13.0)  30.24    466.2  1832.7     FAILED       93.1    33.3  2208.6
 56(14.0)  30.32    477.9  1883.1     FAILED      128.4    46.0  2360.5
 60(15.0)  30.42    493.3  1935.6     FAILED      176.9    63.6  2515.8
 64(16.0)  30.57    513.2  1989.1     FAILED      243.8    87.7  2673.8
 68(17.0)  30.75    538.7  2042.5     FAILED      335.7   120.9  2833.1
 72(18.0)  31.00    571.0  2093.9     FAILED.     462.3   166.6  2991.6
 76(19.0)  31.32    611.3  2140.1     FAILED      636.5   229.5  3145.4
 80(20.0)  31.74    660.8  2176.5     FAILED      876.4   316.1  3288.9
 84(21.0)  32.30    721.2  2195.9     FAILED     1206.5   435.3  3413.1
 88(22.0)  33.02    794.0  2187.2     FAILED     1660.8   599.4  3504.9
 92(23.0)  33.97    881.1  2133.3     FAILED     2286.3   825.3  3544.6
 96(24.0)  35.21    984.9  2007.6     FAILED     3147.1  1136.1  3502.9
100(25.0)  36.86   1107.8  1770.0     FAILED     4332.1  1564.0  3336.9
104(26.0)  39.07   1252.1  1359.3     FAILED     5963.1  2153.0  2982.6
108(27.0)  39.89   1293.3  1166.1  NOT FAILED    4522.8  2153.0  2750.8
112(28.0)  40.11   1293.3  1092.4  NOT FAILED    4370.1  2153.0  2608.5
116(29.0)  40.30   1293.3  1029.8  NOT FAILED    4240.6  2153.0  2488.8
120(30.0)  40.47   1293.3   975.9  NOT FAILED    4129.5  2153.0  2387.0
124(31.0)  40.63   1293.3   928.7  NOT FAILED    4032.3  2153.0  2298.6
128(32.0)  40.77   1293.3   886.8  NOT FAILED    3946.0  2153.0  2220.8
132(33.0)  40.91   1293.3   849.0  NOT FAILED    3868.6  2153.0  2151.1
136(34.0)  41.03   1293.3   814.8  NOT FAILED    3798.4  2153.0  2088.4
140(35.0)  41.14   1293.3   783.5  NOT FAILED    3734.2  2153.0  2031.4
144(36.0)  41.25   1293.3   754.7  NOT FAILED    3675.4  2153.0  1979.2
148(37.0)  41.35   1293.3   728.0  NOT FAILED    3621.0  2153.0  1931.3
152(38.0)  41.44   1293.3   703.2  NOT FAILED    3570.4  2153.0  1886.9
156(39.0)  41.53   1293.3   680.1  NOT FAILED    3523.2  2153.0  1845.6
160(40.0)  41.62   1293.3   658.4  NOT FAILED    3479.1  2153.0  1807.1
164(41.0)  41.70   1293.3   638.0  NOT FAILED    3437.6  2153.0  1771.0
168(42.0)  41.78   1293.3   618.8  NOT FAILED    3398.6  2153.0  1737.2
172(43.0)  41.85   1293.3   600.7  NOT FAILED    3361.7  2153.0  1705.2

BRITTLE/DUCTILE TRANSITION POINT:TAUF=   1298.1
FRACTURE ZONE ENDED ON 105TH SLICE AND EXTENDED 26.3 FT INTO THE PILLAR.
DEPTH OF FRACTURE ZONE JUST AT FAILURE 25.21
```

TABLE A1: STRESS DISTRIBUTION ACROSS PILLARS A, B AND C

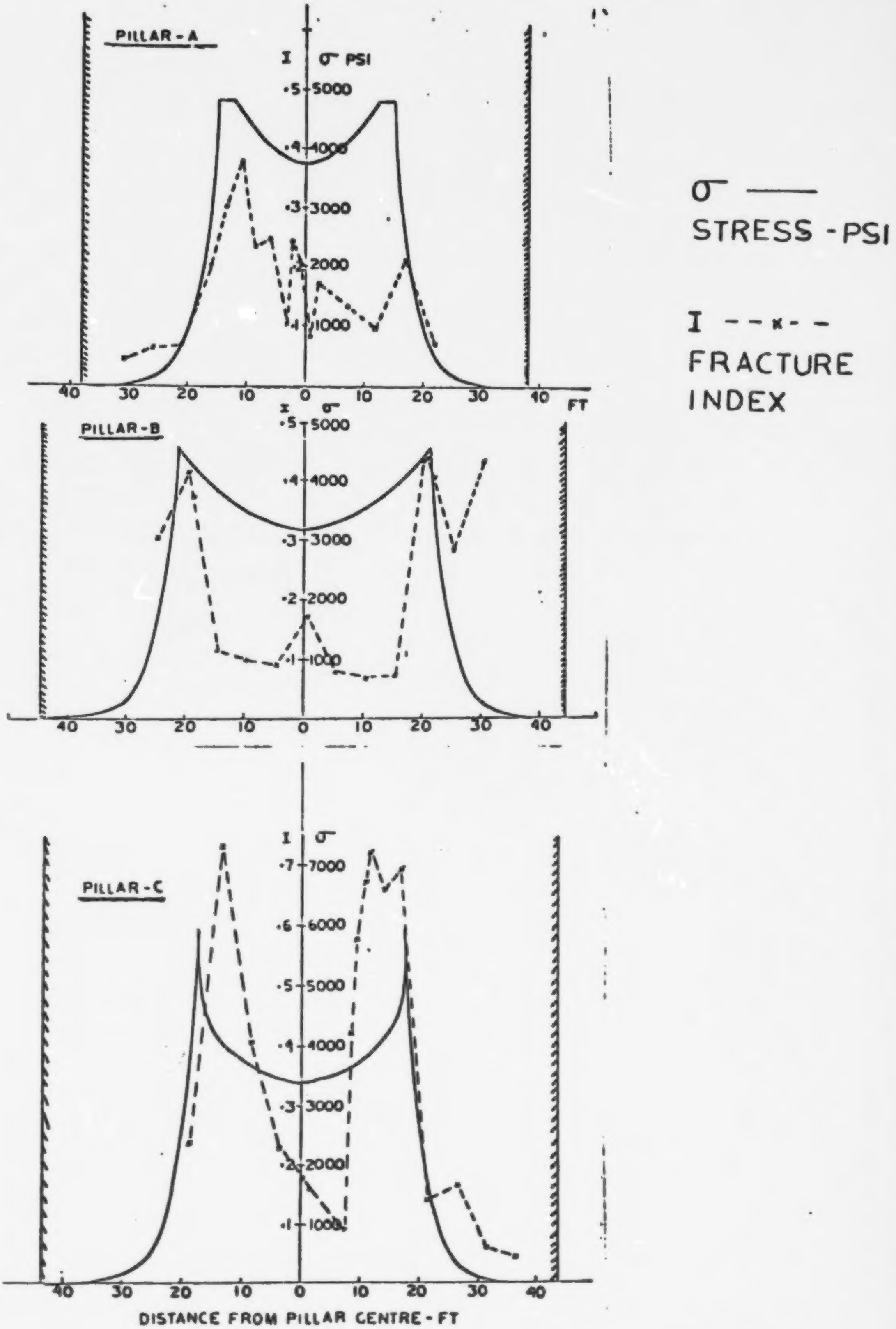

FIGURE A1: COMPARISON OF ANALYTICAL STRESS DISTRIBUTION WITH MEASURED FRACTURE INDEX - 3 PILLARS IN #4 SEAM

APPENDIX 3-5

AN EVALUATION OF THE SHEARING OF THE NO. 4 SEAM IN THE LONGWALL MINING DEVELOPMENT AREA

NORWEST RESOURCE CONSULTANTS

AN EVALUATION OF THE SHEARING OF THE NO. 4 SEAM IN THE LONGWALL MINING DEVELOPMENT AREA

THE ASSOCIATION OF
PROFESSIONAL ENGINEERS,
GEOLOGISTS and GEOPHYSICISTS
OF ALBERTA
PERMIT NUMBER
P 914
NORWEST RESOURCE
CONSULTANTS LTD.

Prepared For: McIntyre Mines
Prepared By: Norwest Resource Consultants Ltd.

Report No.: 82-235/2

October, 1982

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APPENDIX A SHEAR ZONE SEAM SECTIONS

LIST OF MAPS

(In Map Pockets)

MAP 1 ISOPACH MAP - PRINCIPAL LOWER SHEAR ZONE
MAP 2 ISOPACH MAP - TOTAL SHEAR ZONE
MAP 3 ISORATIO MAP - PRINCIPAL LOWER SHEAR ZONE VERSUS TOTAL SHEAR THICKNESS
MAP 4 ISORATIO MAP - TOTAL SHEAR ZONE VERSUS TOTAL SEAM THICKNESS
MAP 5 ISOPACH MAP - THICKNESS OF SOLID COAL FLOOR

SUMMARY

An evaluation of the zone of shearing of the No. 4 Seam in the area planned for longwall development was carried out at the request of McIntyre Mines by Norwest Resource Consultants Ltd. Twenty-five drill hole geophysical logs and one underground seam exposure were used in this evaluation. The following points summarise the method of shear zone detection developed in this study:

1) Geophysical logs can be used to determine the location and thickness of zones of coal shearing.
2) Focused resistivity and caliper logs are the most useful for this purpose.
3) Density logs occasionally produce a pertinent response.
4) The reliability of detection depends upon the intensity of shearing and the amount of water contained in the shear zone.
5) The resistivity of sheared coal becomes anomalously low due to the water in the shear zone.
6) The caliper log response changes at shear zones due to caving or differential hole closure.
7) The density of the zone may be anomolously low due to the tectonically induced high porosity and shear zone drainage.

A series of isopach maps were prepared to evaluate the results at each data point. The maps are:

1) The thickness of the lower principal shear zone.
2) The percentage of the lower principal shear zone with respect to the total seam thickness.
3) The total thickness of sheared coal.
4) The percentage of the total seam shearing with respect to the total seam thickness.
5) The thickness of solid, essentially unsheared coal between the top of the lower principal shear zone and the base of a theoretical, 10 feet thick mining section extracted from the top of the seam.

The first four maps listed above were contoured with a subjective bias to take into account seam structures related to the large scale geological structures. Biased contouring was necessary because of the large number of structural domains and insufficient data points to allow representative random contouring of each of them.

The last of the maps listed above includes no subjective bias. The most significant results are listed as follows:

1) The data at each point created no anomalous areas with respect to the biased contouring.
2) As a result, the theoretical considerations used to establish the contouring bias are assumed to be more or less appropriate.
3) As many as four significant coal shear zones are present in any one seam intersection.
4) The number of zones present depends upon the degree of tectonic deformation; the greater the degree of deformation the greater the number of shear zones present.
5) One shear zone, the lower principal shear zone is always present.
6) The number of zones increases as the strata change from essentially flat lying, to relatively steeply dipping and then to synclinal, anticlinal or monoclinal folding with a significant amount of rotation of the strata.
7) Only the lower principal shear zone will have an impact on the performance of roof supports standing on the floor of a 10 feet mining section since the other zones would almost always be extracted in the mining section.
8) The thickness of solid coal between the top of the lower principal shear zone and the floor of the ten foot thick mining section will almost always be greater than 3 feet thick and average four feet thickness in the planned longwall area.
9) The axial region of the Taylor Syncline is likely to pose problems for mining due to this sheared coal factor.
10) The No. 4 Seam in the axial region of the Taylor Syncline appears to be more intensely deformed than in the axial region of the Drill Camp Anticline.

11) Any expansion of the planned development would involve less risk due to the shearing if those expansions were developed further west along the structure than north or south across the Taylor Syncline or the Drill Camp Anticline.

12) Most of the area planned for development is expected to have similar conditions to those already displayed by the full seam underground exposure. Variations of significance may occur in the region of drill holes No. 80-05, 80-08 and 80-10 and the axial region of the Taylor Syncline.

13) The area of drill holes 80-05, 80-08 and 80-10 has an unusually thick lower principal sheared zone but problems for roof supports may not occur because the thickness of solid coal floor would still be three feet thick or greater.

1. INTRODUCTION

At the request of McIntyre Mines Ltd. an evaluation of borehole geophysical logs drilled in the No. 9 Mine longwall development area was carried out with a view to investigating soft sheared coal sections in the No. 4 Seam. This evaluation was undertaken because a preliminary study of the shear zone suggested that a thorough investigation of all of the geophysical data could be used to document the distribution and thickness of that material on a relatively detailed scale suitable to aid mine planning. This evaluation was possible because it appeared that the zones of shearing produced a response to soft sheared coal on some of the geophysical logs.

A total of 26 data points included all of the necessary information for interpretation, and the following report includes a discussion of the theoretical considerations used in the interpretation, the methodology applied, and the results produced. Numerous drill hole geophysical log sections and several maps are included showing the distribution of the sheared coal material.

It is believed that the results of this investigation will be suitable to account for the distribution of the soft sheared coal in No. 4 Seam. An understanding in greater detail of the distribution and the impact of this material on longwall mining can be further developed by in-mine investigations as development work proceeds.

2. THEORETICAL CONSIDERATIONS FOR SHEAR ZONE EVALUATION

It is common to find throughout the mountains and foothills coal regions of western Canada that coal seams show intense levels of tectonic disturbance producing zones of shearing within the coal seams. Material within these zones consists of very small coal fragments and rock which is slickensided on all surfaces. Each fragment is often about 1 centimeter in size and the fragments are usually not bonded together by material such as calcite or other forms of cement. Often these zones are water bearing and exposed faces show water issuing from the fractures between the coal fragments.

When seam sections such as this are drilled, the sheared material tends to cave into the drill hole because of its exceptionally friable nature. Once the drilling operations are completed the sheared coal may tend to flow into the hole more rapidly than hole closure takes place for the adjacent strata. As a result, these zones can often be detected, in part or in whole, by a noticeable response on caliper logs showing a change of hole diameter with respect to the surrounding coal or rock strata. The response on the caliper log may show either a larger hole diameter or a smaller hole diameter through the zone of sheared coal.

Density geophysical logs do not usually show a change of response through the sheared coal zone but, on occasion, if the sheared zone is allowed to drain, the dry coal section will cause an unusually low density response due to the increased porosity caused by the shearing action. Gamma-ray logs rarely provide any information to suggest the presence of zones of shearing of coal.

The most important response for the detection of zones of coal shearing occurs on focused resistivity logs. These logs have an exceptionally high resolution capability and are particularly suitable for detailed investigation of coal seams. They also have a strong response to wet coal plys including wet sheared zones. The resistivity drops markedly through these wet coal horizons.

By comparing the response on caliper logs, on focused resistivity logs, and occasionally on density geophysical logs it is possible to determine the location and thickness of sheared coal. The reliability of interpretation depends on the level to which the coal is sheared and the extent that the zone is water bearing.

In the No. 9 mine area several zones of shearing of the No. 4 Seam occur. These zones are not necessarily continuous laterally but there appears to be a fairly uniform lateral distribution of at least some of them. In the least tectonically disturbed areas at least one zone of shearing is always present and in the most disturbed areas that zone, plus as many as three others, appear to be present. The zone of shearing that is always present is the most significant one for mine development because it occurs towards the base of the seam and has a direct impact on the performance of roof supports located above this material. The other zones of shearing have a lesser impact because they appear to show a lower level of tectonic disturbance and they would normally be located within a ten foot mining section extracted from the upper part of the seam.

3. METHODOLOGY

Twenty-five drill hole logs and one underground exposure were used to develop the evaluation of the sheared zone of No. 4 Seam. For each drill hole gamma-ray, density, caliper and the focused resistivity logs of the full seam were used. Zones of unusually low resisitivity were located on the focused resisitivity log and these seam intervals were compared with the caliper log response through that zone to determine whether an anomalous response also occured on those logs. In one instance an unusually low density response was detected. All of this information was combined and the logged seam sections were marked according to the location of shear zones. Different levels of confidence in the interpretation of the zones was also indicated. The different levels of confidence are shown by "possible", "probable", or "shear zone" on the accompanying seam sections.

The results of thicknesses were tabulated and a series of isopach maps produced. Isopach maps No.'s 1 and 2 show the thickness of the principal lower sheared coal horizon and the thickness of the total amount of sheared coal respectively. The contouring is biased to the overall structure of the area because it can be seen in surface and underground exposure as well as drill holes that the intensity of shearing of the seam at any one place is related to the amount of rotation of the strata at that point. This means that in the vicinity of synclines, anticlines or monoclinal axes the amount of shearing of the seam will be greatest and will reduce progressively through relatively steeply dipping strata where a greater extent of bedding plane movements can be predicted, to essentially flat lying strata. It is obvious therefore, that contouring has been conducted on a subjective basis to account for this factor. It is not possible to contour on a random distribution basis and produce a meaningful result because the number of structural domains relative to dip and rotation of the strata are too great to be adequately represented by the number of data points available. It is also not practical to consider producing sufficient data to allow random contouring to be undertaken with this particular problem because the number of drill holes required would be an unreasonably high one. Nevertheless contouring with this subjective bias showed that the information at each data point did not violate the theoretical considerations and therefore it is believed that the original assumptions are more or less appropriate ones. Map No.'s 3 and 4 are contour maps of the shearing of the principal lower zone and the total amount of shearing as a percentage of the seam thickness and they are therefore more meaningful maps to investigate the extent to which the seam is deformed. Map No. 5 is a contour map which has been prepared without subjective bias. This map shows the thickness of unsheared coal lying between the top of the lower principal sheared zone and the base of a theoretical ten foot mining section extracted from the top of the seam.

4. RESULTS OF THE EVALUATION

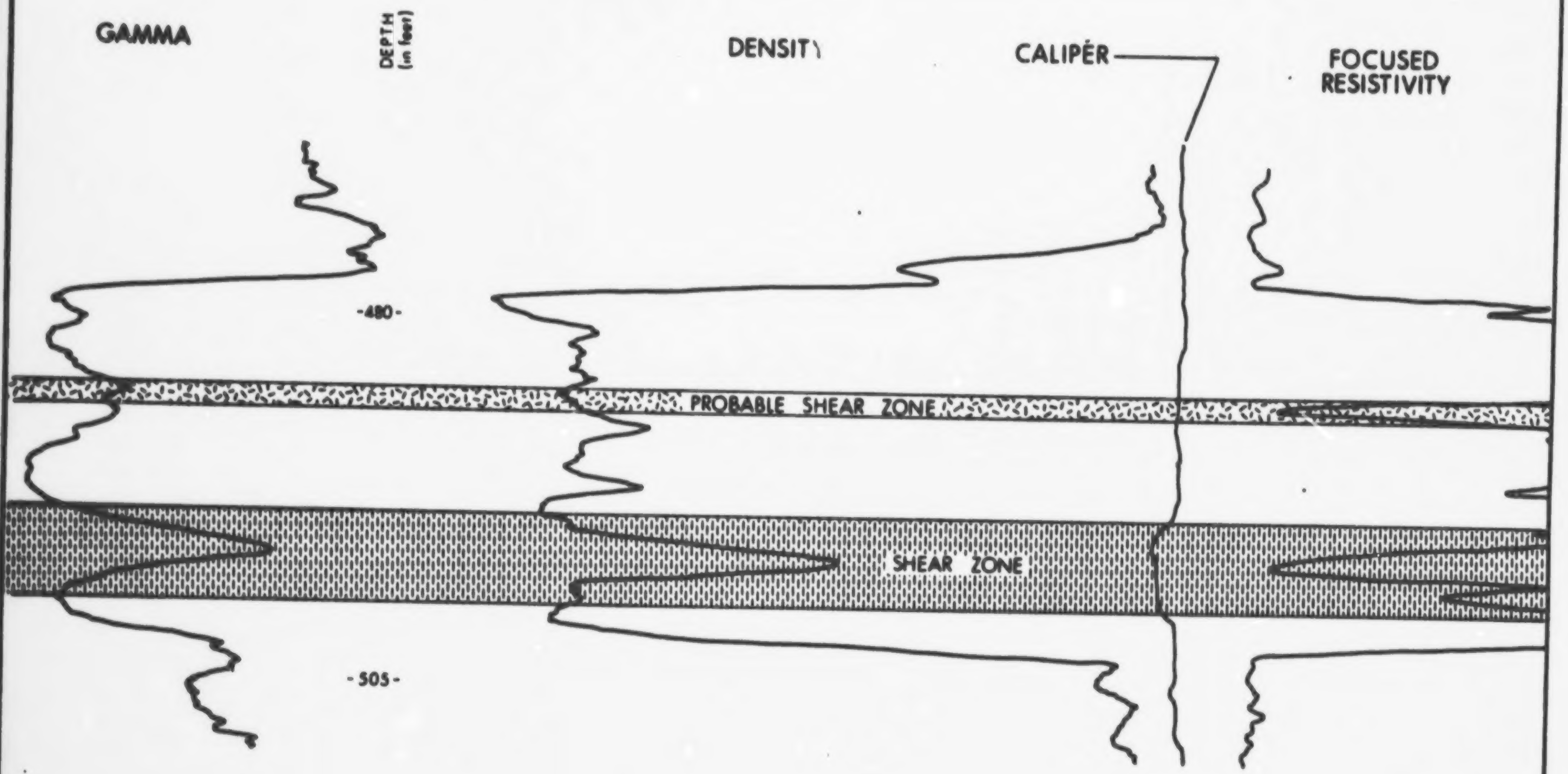
It is believed that a confidence level of about 80% can be applied to the reliability of determination of the thickness and distribution of sheared zones within the seam within any given drill hole. This confidence level of course changes depending upon the extent to which the coal is sheared but the highest level of confidence can be placed on the determination of thickness and distribution of the principal lower sheared zone. Map No.'s 2 and 4, for the total amount of shearing of the seam and its percentage with respect to total seam thickness, show that the amount of shearing is greatest near regions of folded strata including relatively minor flextures of the strata along which monoclinal axes could be plotted.

The results also show that synclinal axes may have greater tendencies to shearing than anticlinal structures. Map No.'s 1 and 3 for the lower principal sheared zone suggest that there is not necessarily a direct relationship between the overall rotation of the strata and the development of the lower sheared zone within the seam, but there are certainly several instances where this occurs.

It should be noted that only one area exists where the lower zone of shearing is predicted to be anomalously thick within the planned area for longwall development. This area is located in the vicinity of drill holes 80-05, 80-10 and 80-08. However, the isopach map of solid coal floor suggests that there is no anomalous change of thickness within this area.

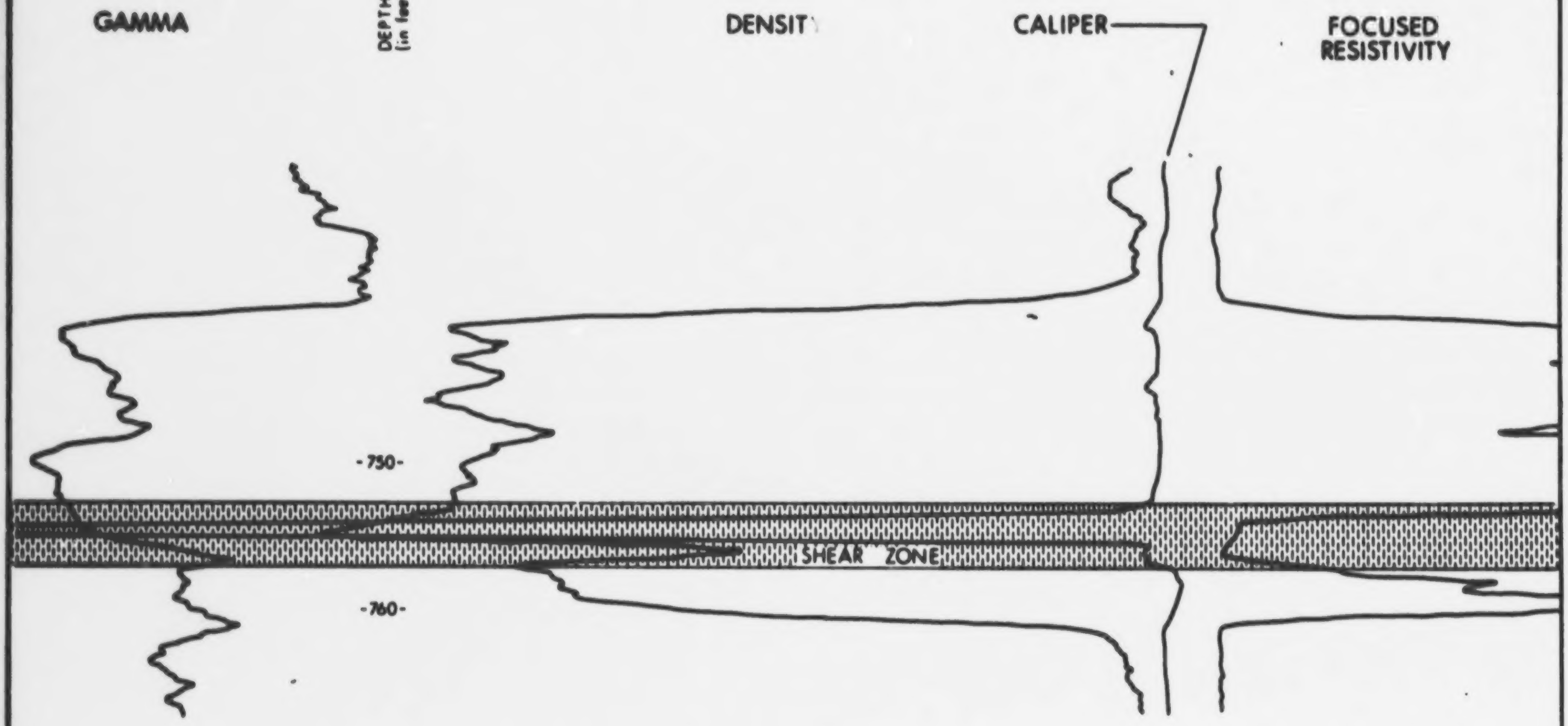
It should be anticipated that the greatest seam shearing changes which might affect mine development will occur in the vicinity of the axial region of the Taylor syncline and caution should therefore be taken when planning development in this region. The solid coal floor map clearly indicates a consistent and even distribution of solid material to the lower principal sheared zone throughout the area planned for the development between the Taylor syncline and Drill Camp Anticline. The map also shows that this thickness changes markedly to the north once the Taylor syncline has been crossed. That map indicates that it is reasonable to assume a section of at least three feet of solid coal beneath the floor of a ten foot mining section and that the average thickness of solid coal will be about four feet.

All of the maps suggest that the amount of deformation producing shearing of the coal is greatest in the vicinity of the Taylor syncline and less in the vicinity of the Drill Camp Anticline and that it may be possible to avoid problems from a soft coal floor more easily in the region of the Drill Camp Anticline then in the Taylor syncline. The maps also suggest that, for an expanded operation, fewer problems from sheared coal in the floor would occur by planning developments further along the structure rather than by planning development panels extending north into the region of the Taylor syncline or south beyond the limit of the Drill Camp Anticline.



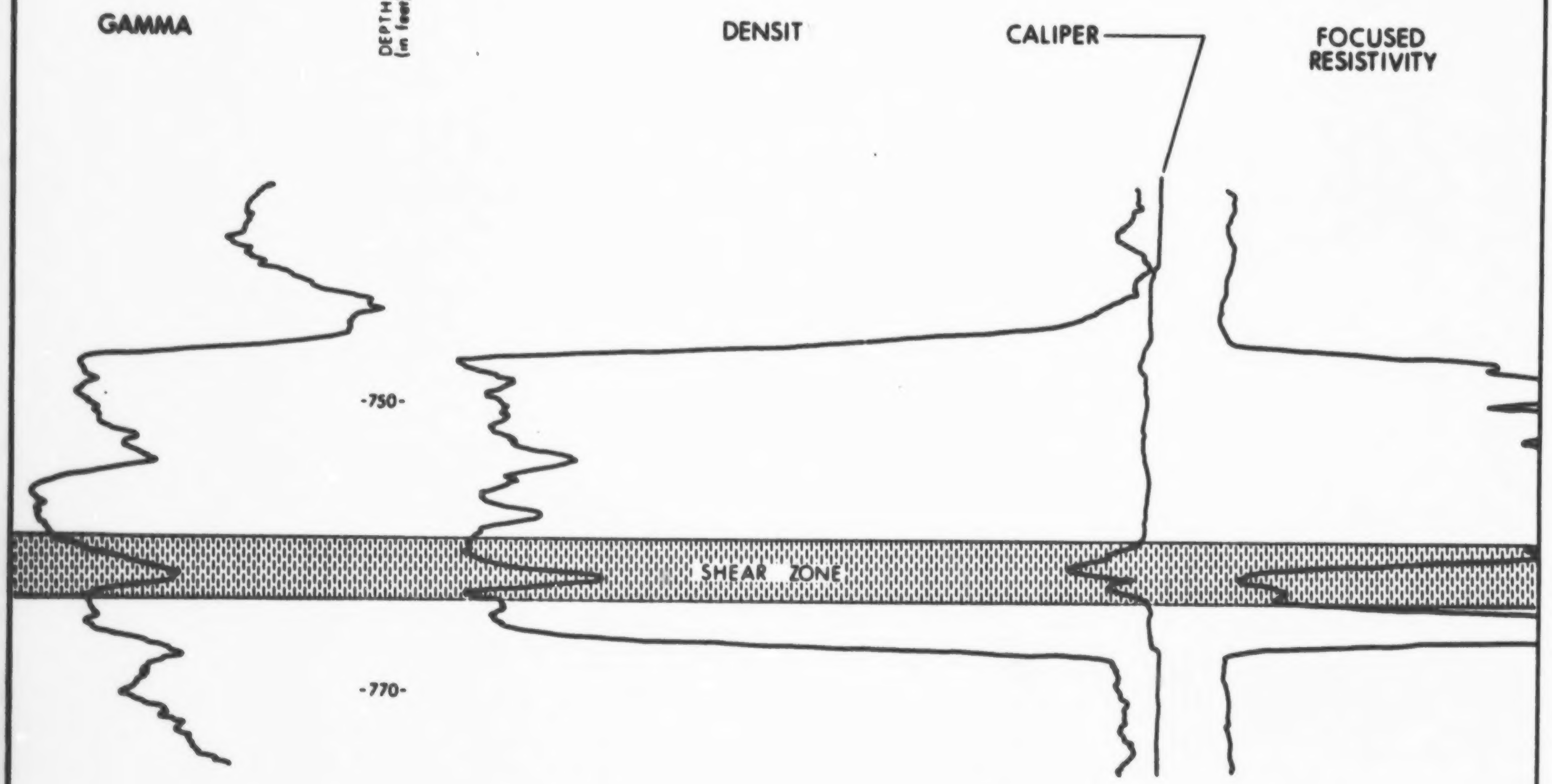
DRILLHOLE No. 76 - 118
SHEAR ZONE No. 4 SEAM

FIGURE No. 1



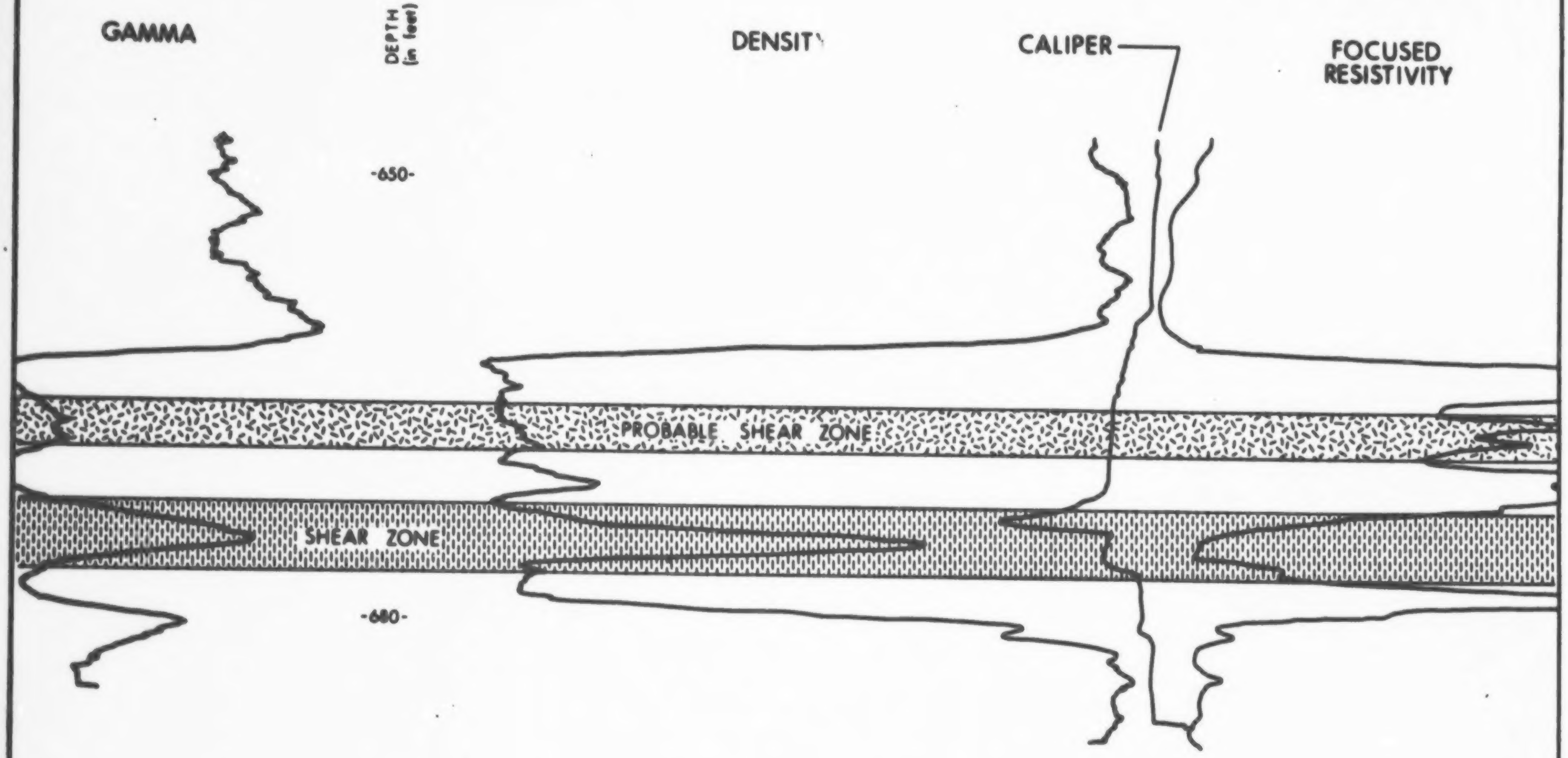
DRILLHOLE No. 76-128
SHEAR ZONE No. 4 SEAM

FIGURE No. 2



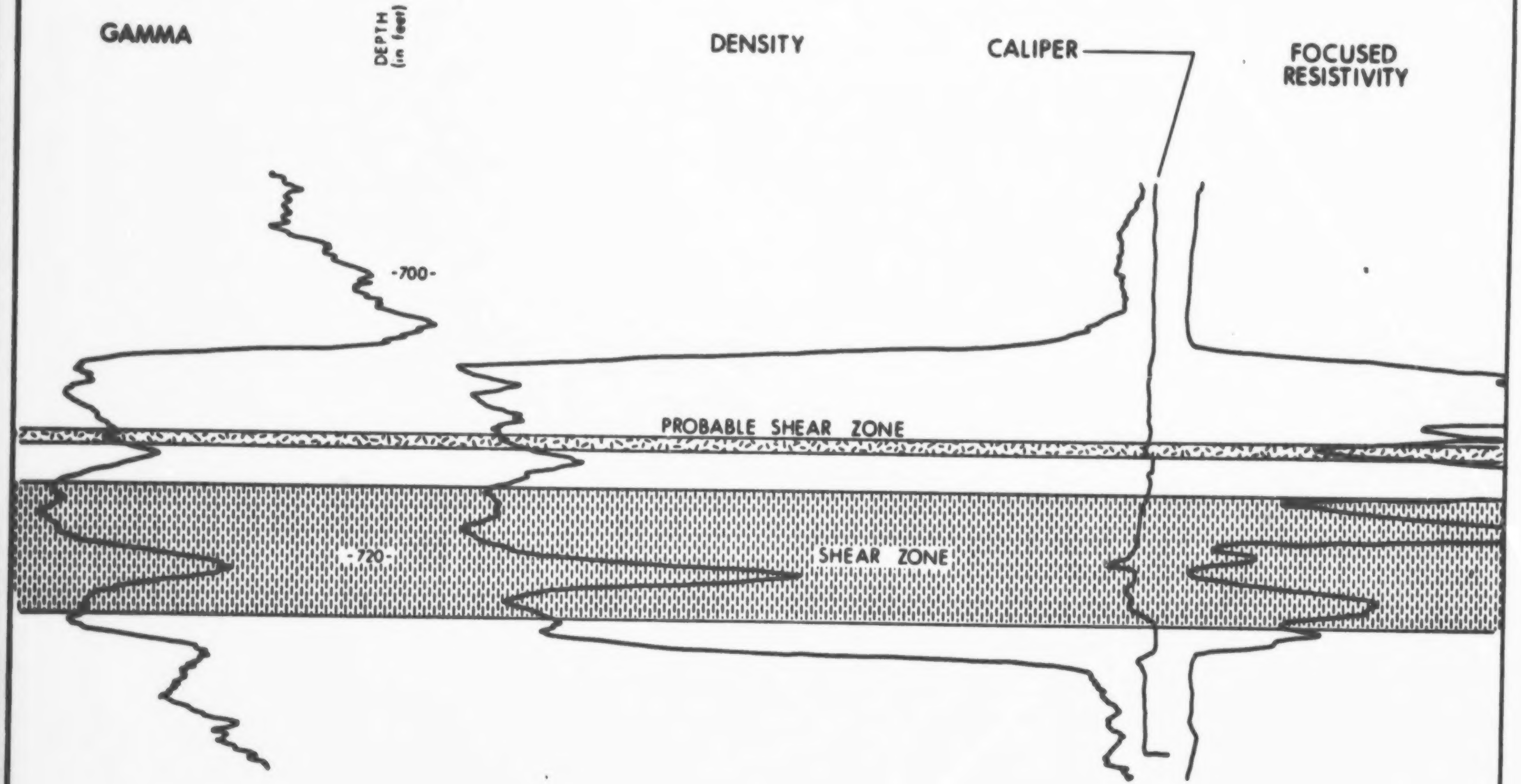
DRILLHOLE No. 76-130
SHEAR ZONE No. 4 SEAM

FIGURE No. 3



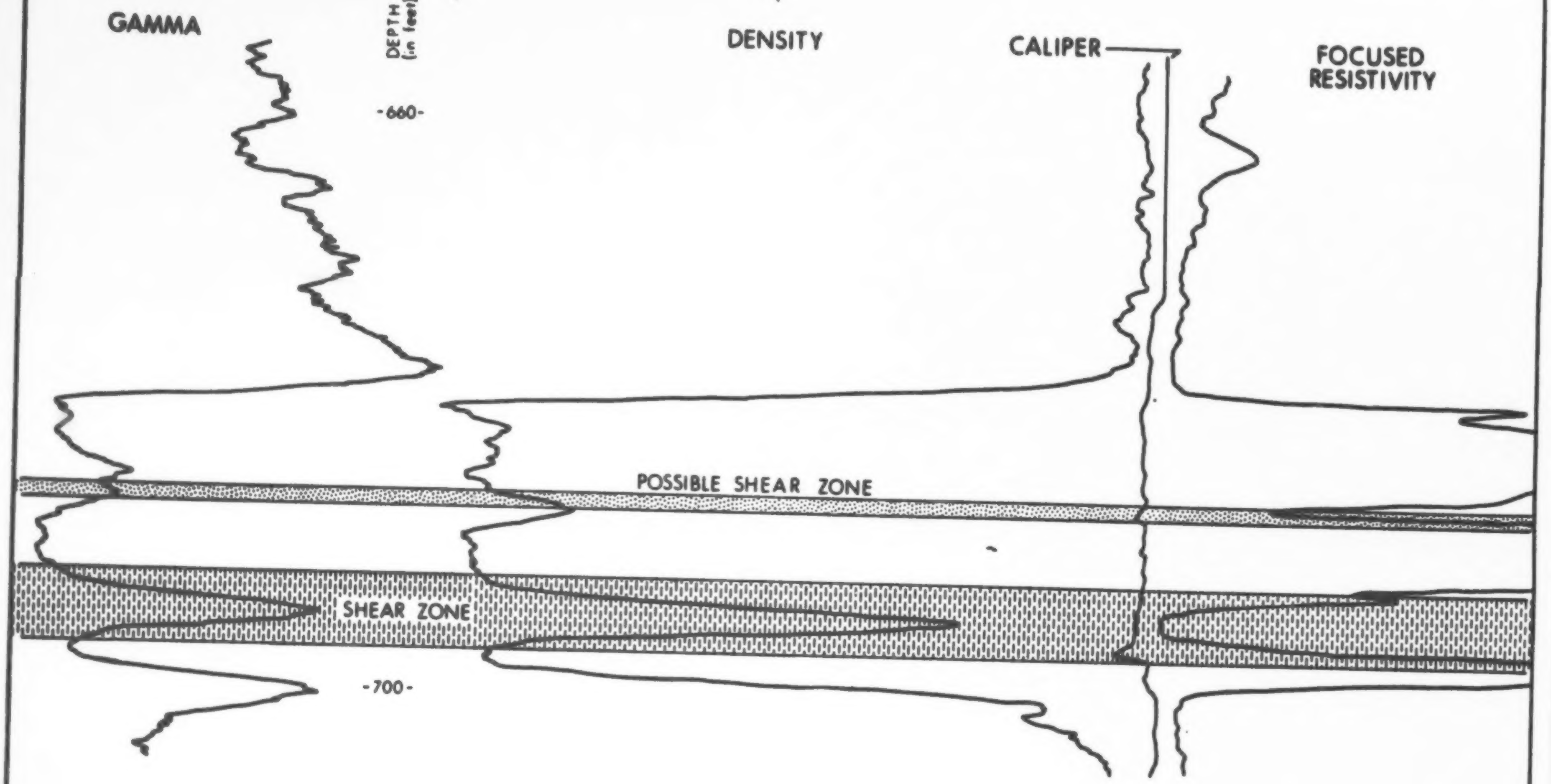
DRILLHOLE No. 76-131
SHEAR ZONE No. 4 SEAM

FIGURE No. 4



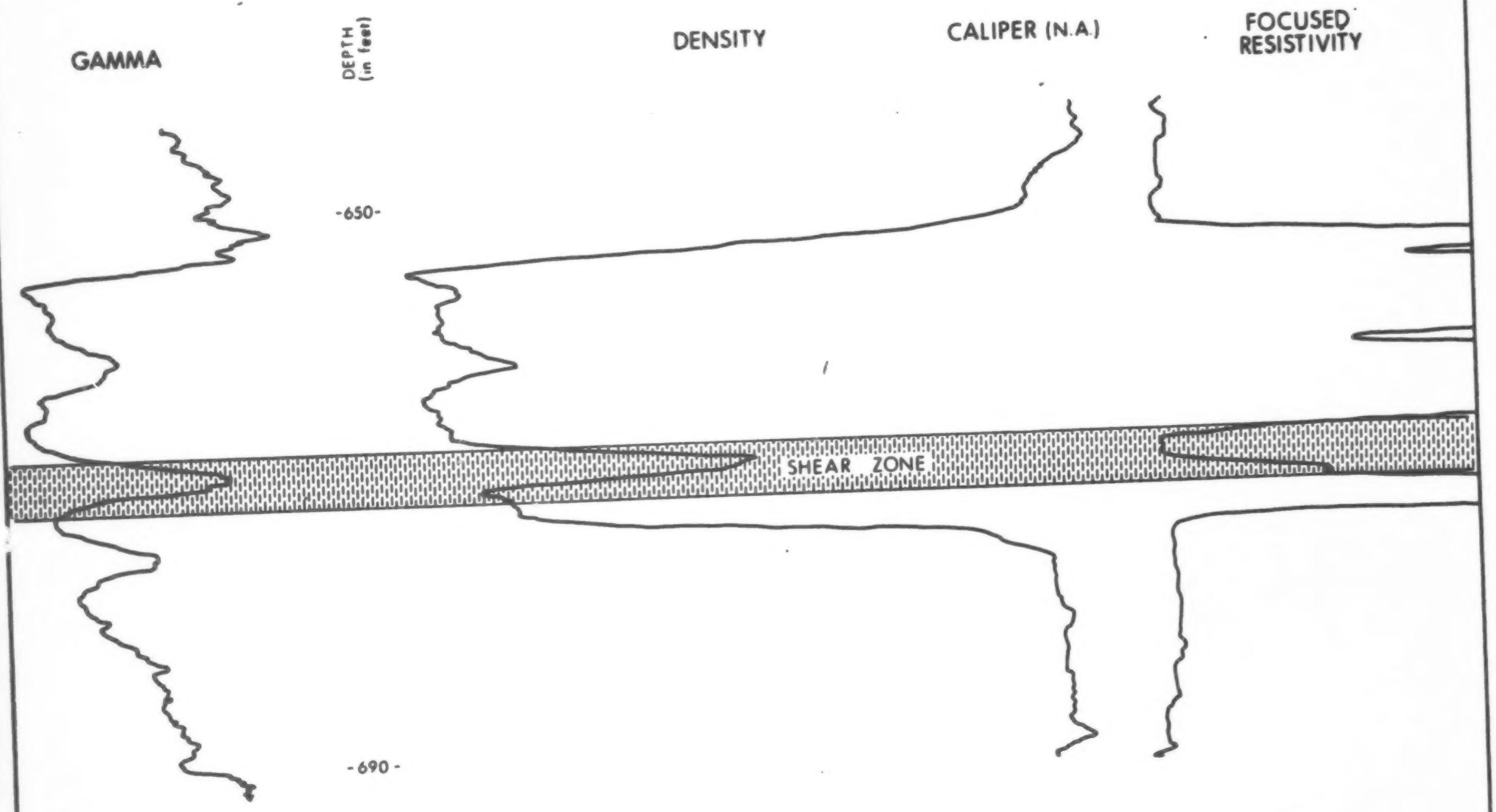
DRILLHOLE No. 76-138
SHEAR ZONE No. 4 SEAM

FIGURE No. 5



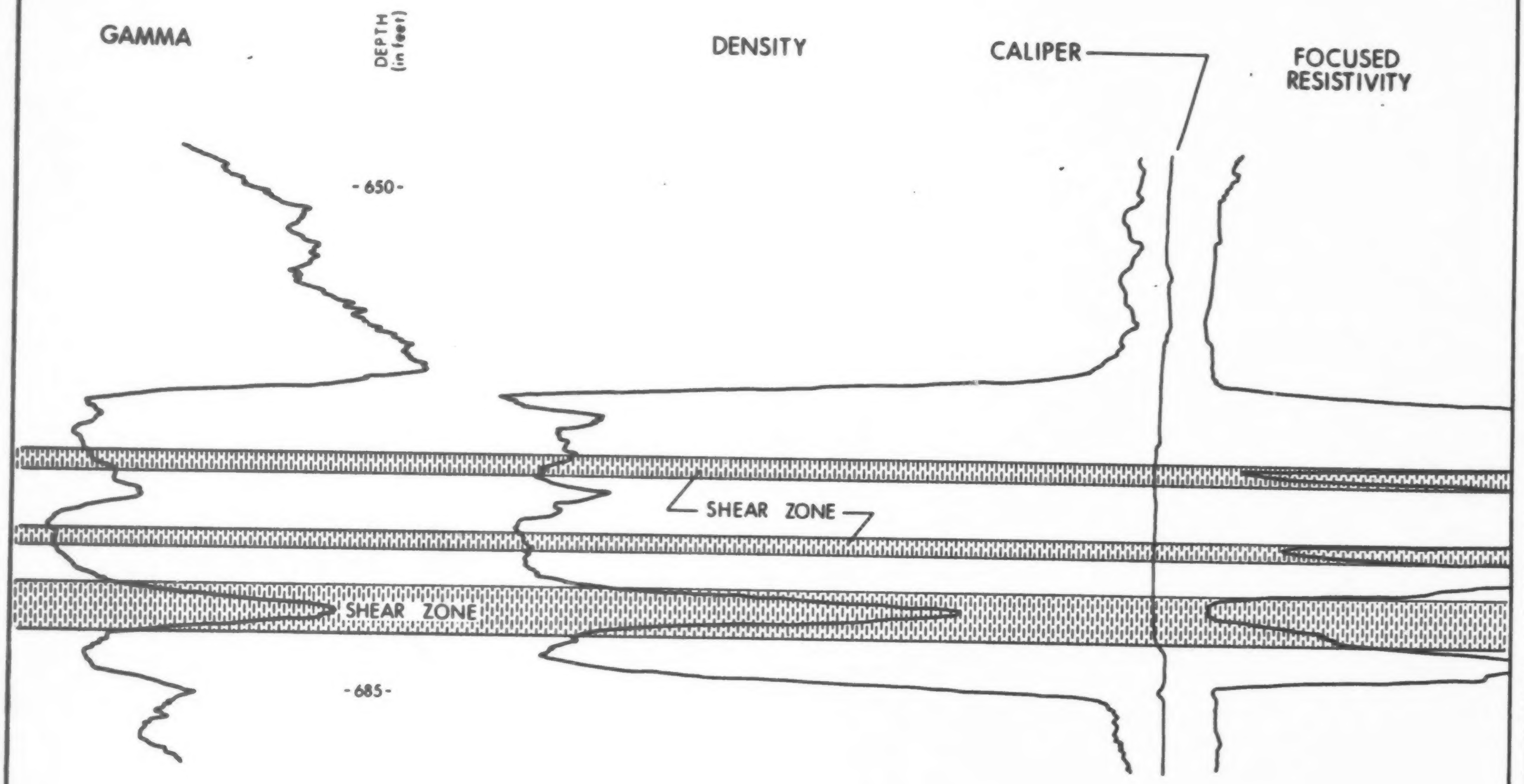
DRILLHOLE No. 76-149
SHEAR ZONE No. 4 SEAM

FIGURE No. 6



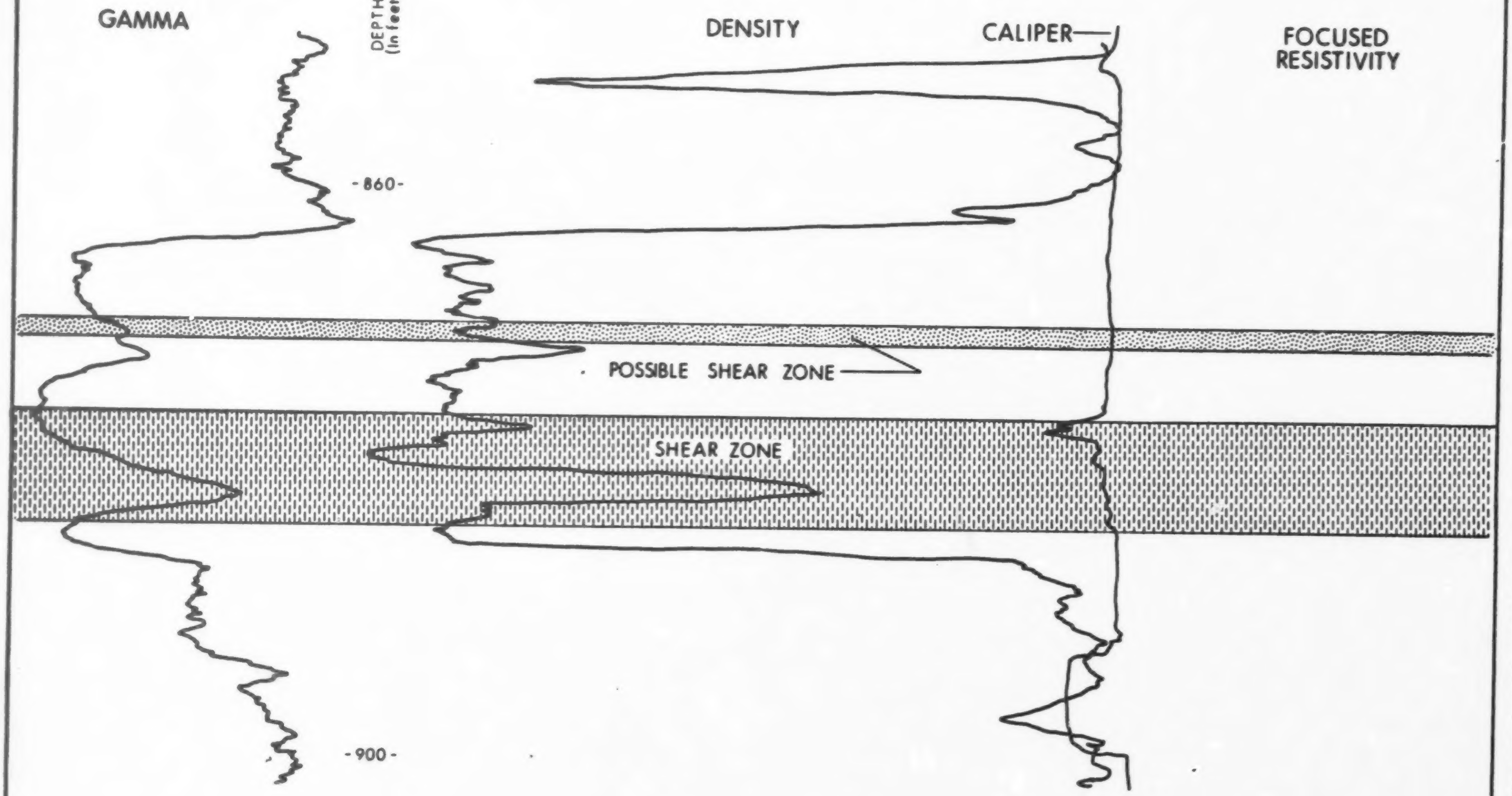
DRILLHOLE No. 76-150
SHEAR ZONE No. 4 SEAM

FIGURE No. 7



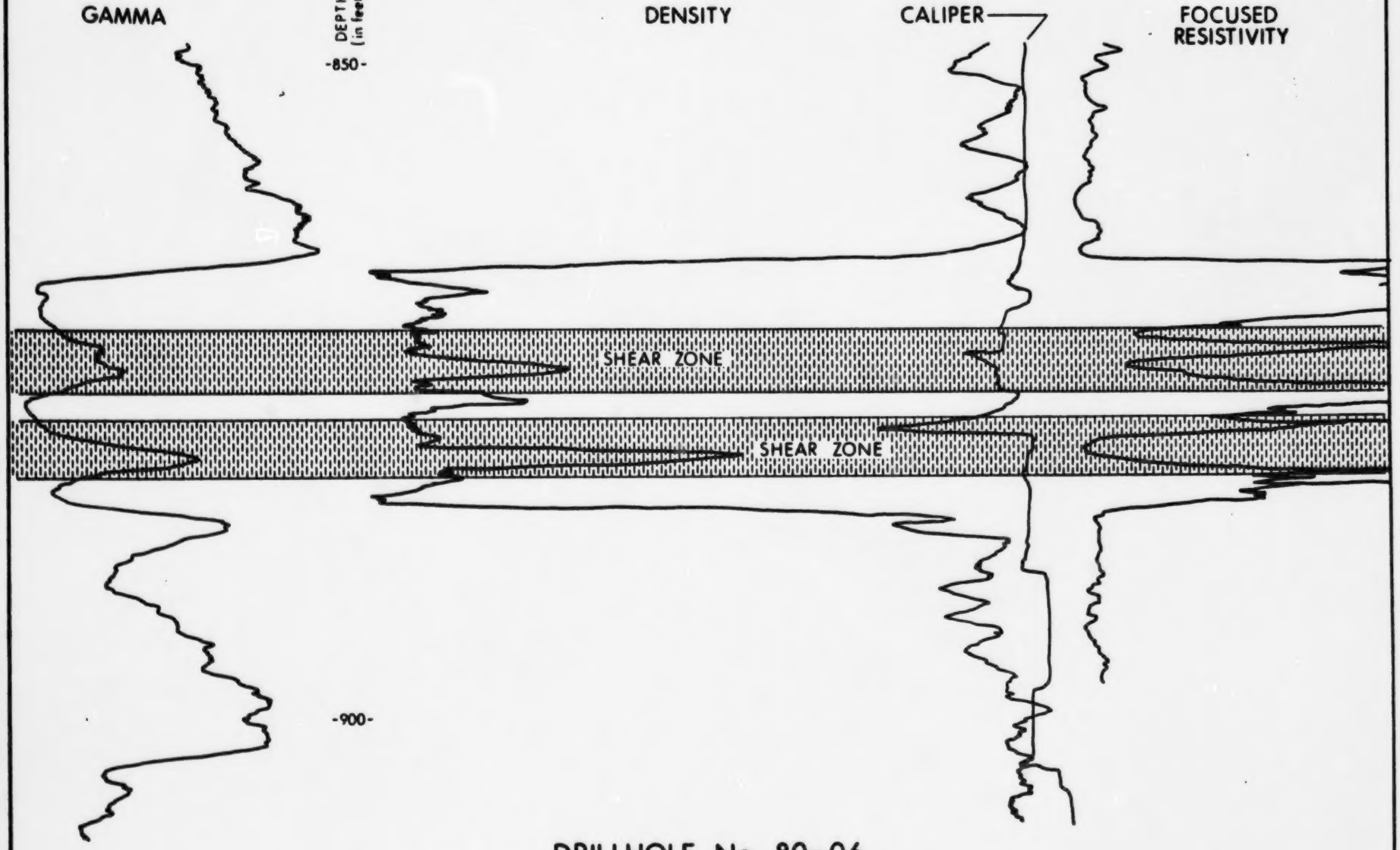
DRILLHOLE No. 77-69
SHEAR ZONE No. 4 SEAM

FIGURE No. 8



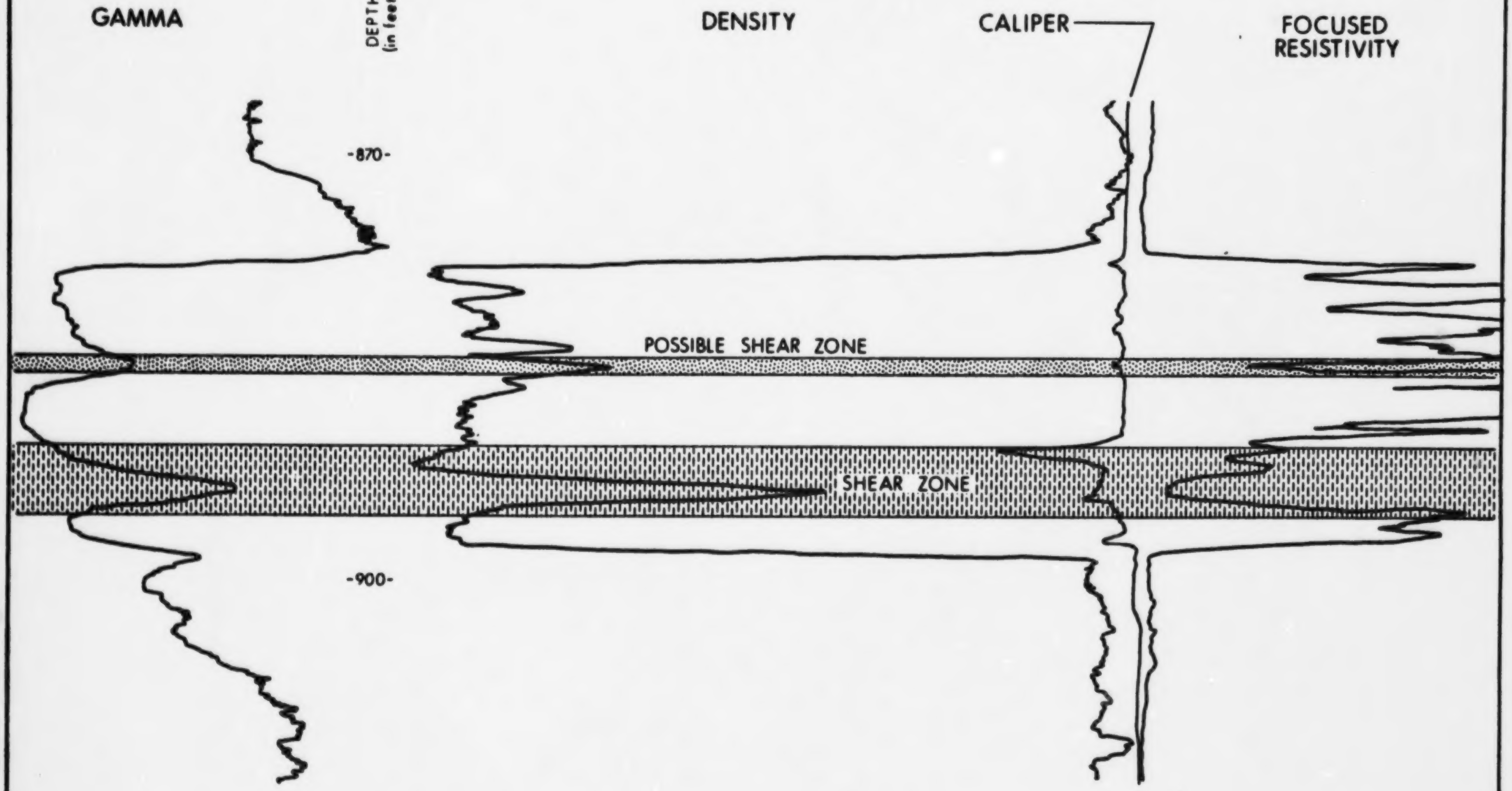
DRILLHOLE No. 80-05
SHEAR ZONE No. 4 SEAM

FIGURE No. 9



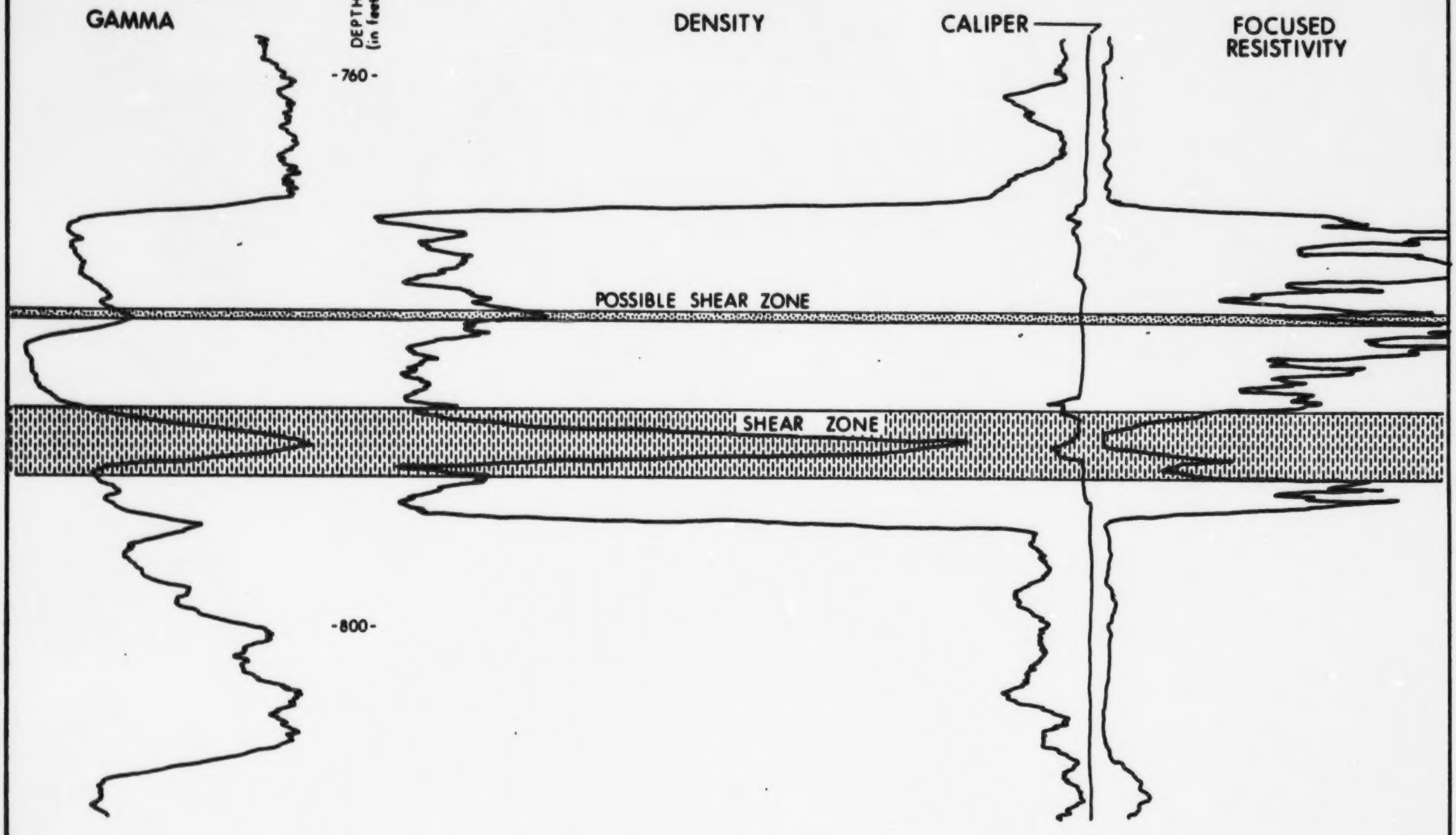
DRILLHOLE No. 80-06
SHEAR ZONE No. 4 SEAM

FIGURE No. 10



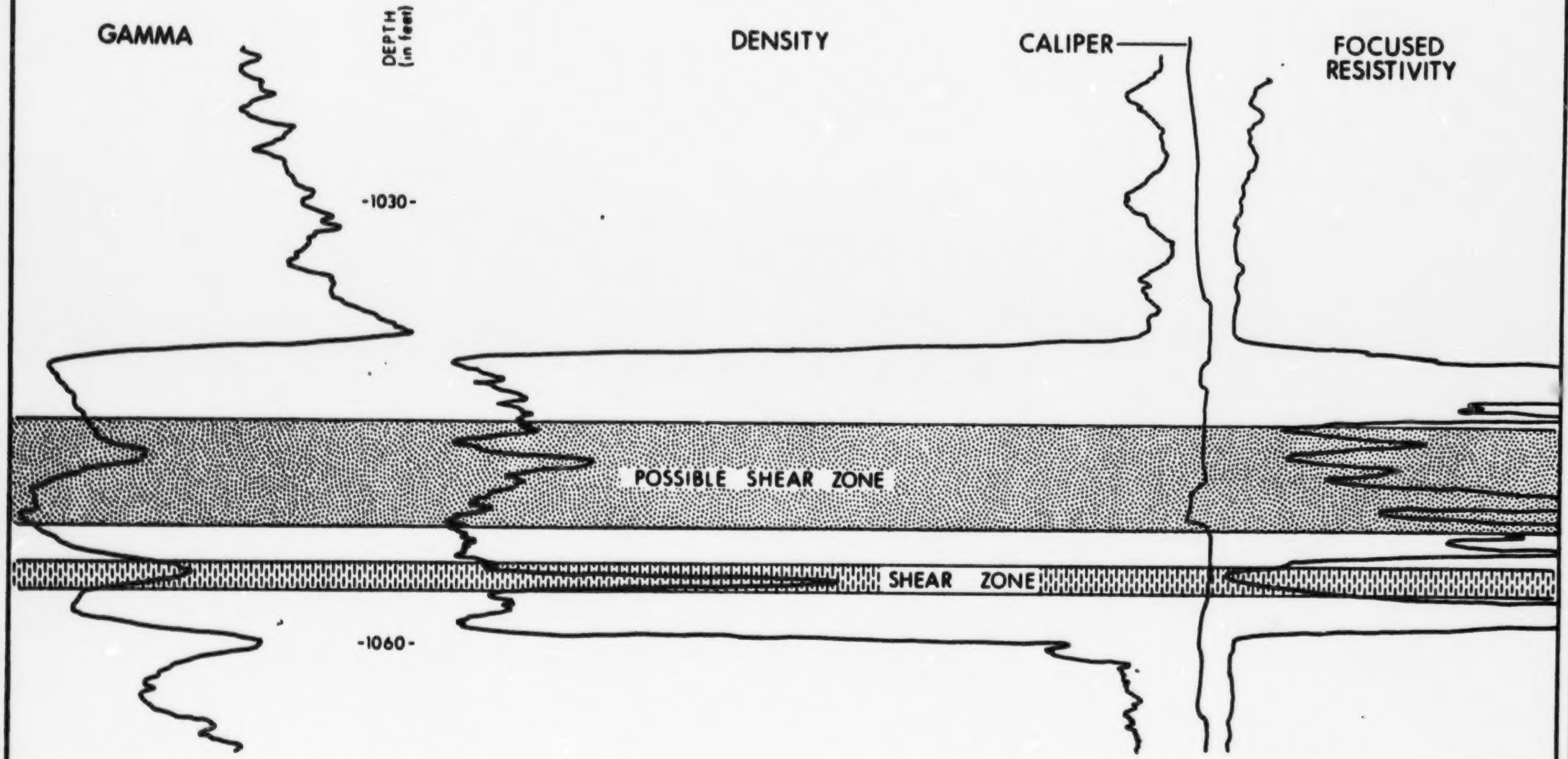
DRILLHOLE No. 80-07
SHEAR ZONE No. 4 SEAM

FIGURE No. 11



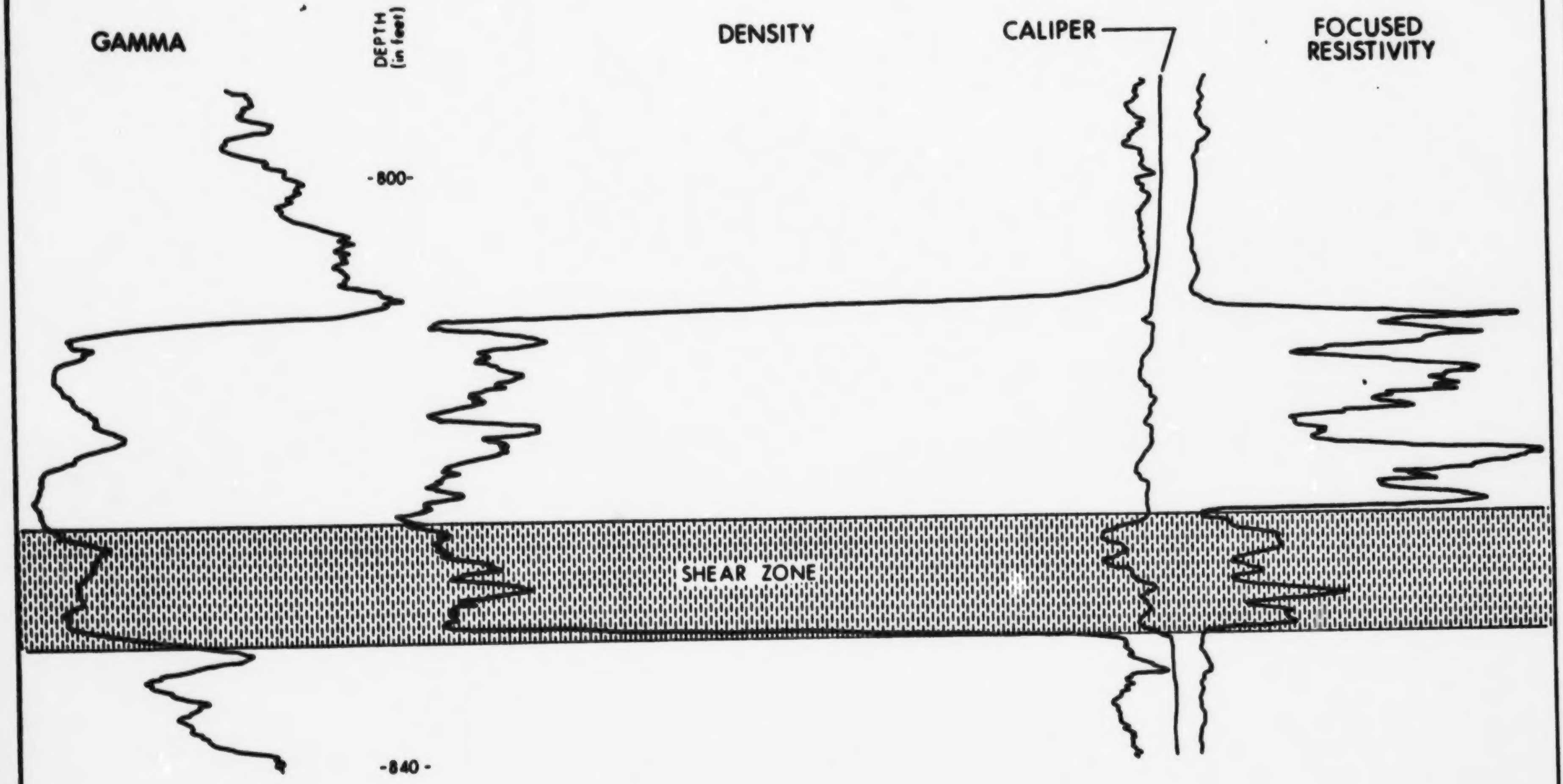
DRILLHOLE No. 80-08
SHEAR ZONE No. 4 SEAM

FIGURE No. 12



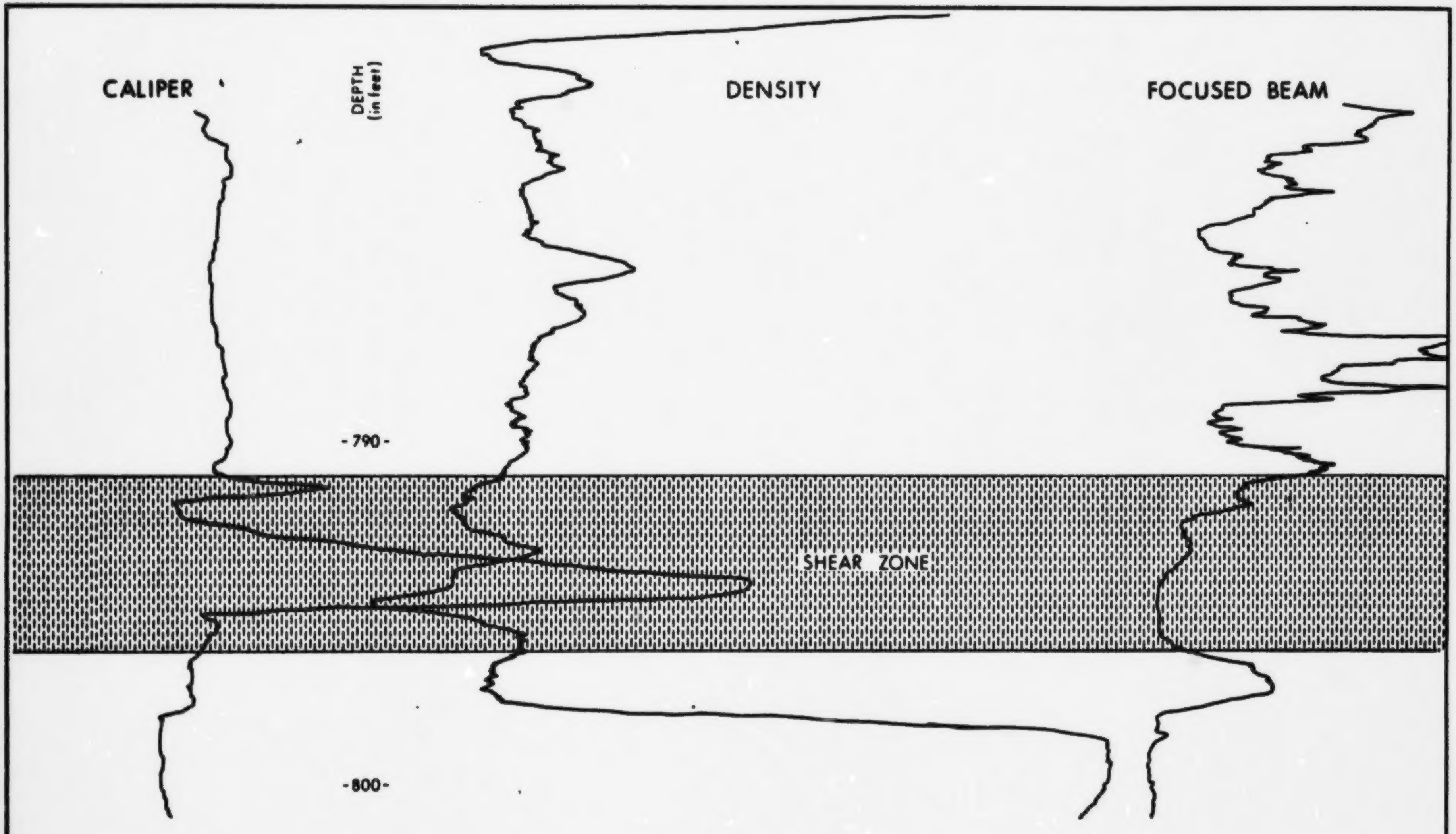
DRILLHOLE No. 80-09
SHEAR ZONE No. 4 SEAM

FIGURE No. 13



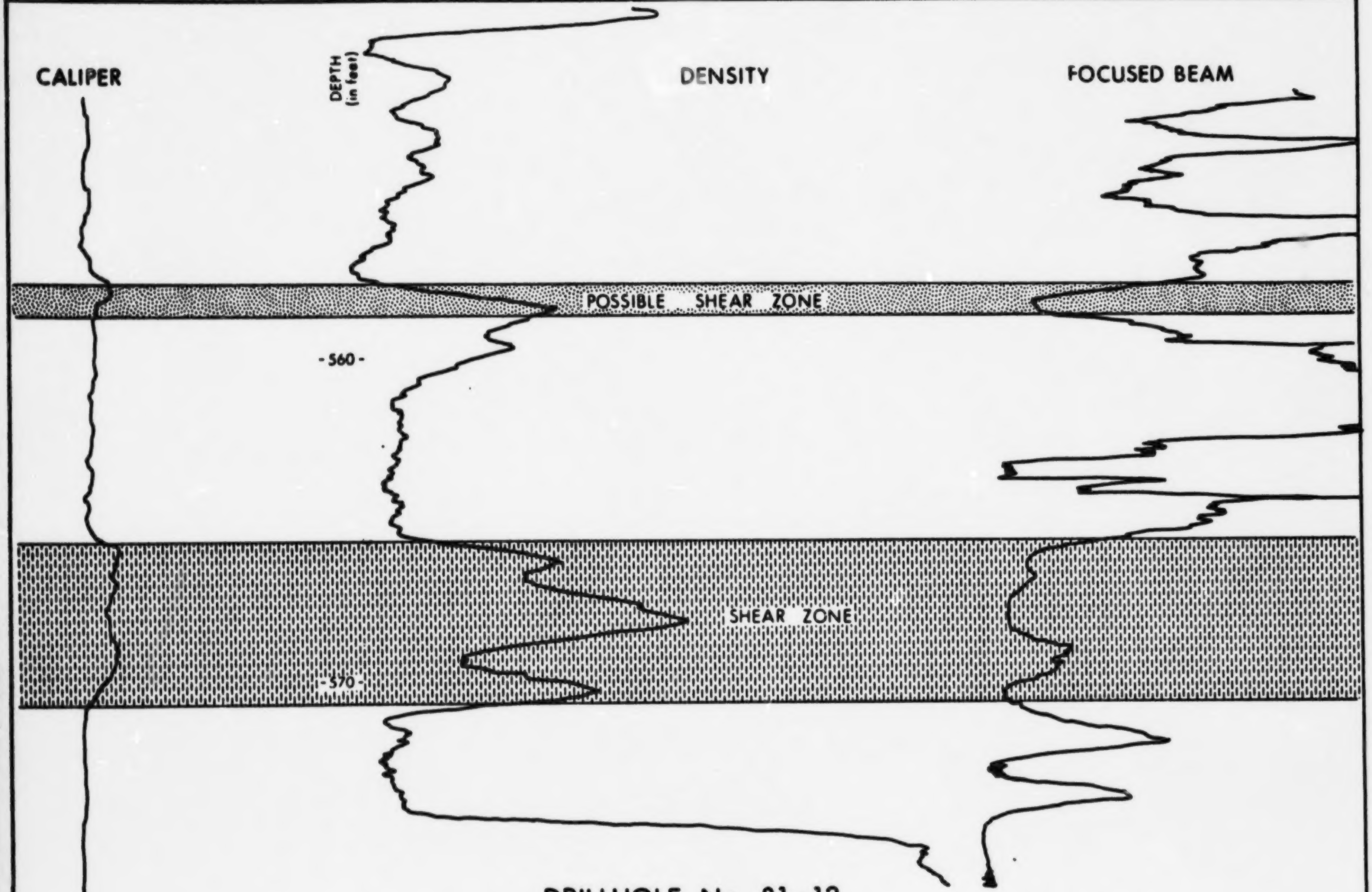
DRILLHOLE No. 80-10
SHEAR ZONE No. 4 SEAM

FIGURE No. 14



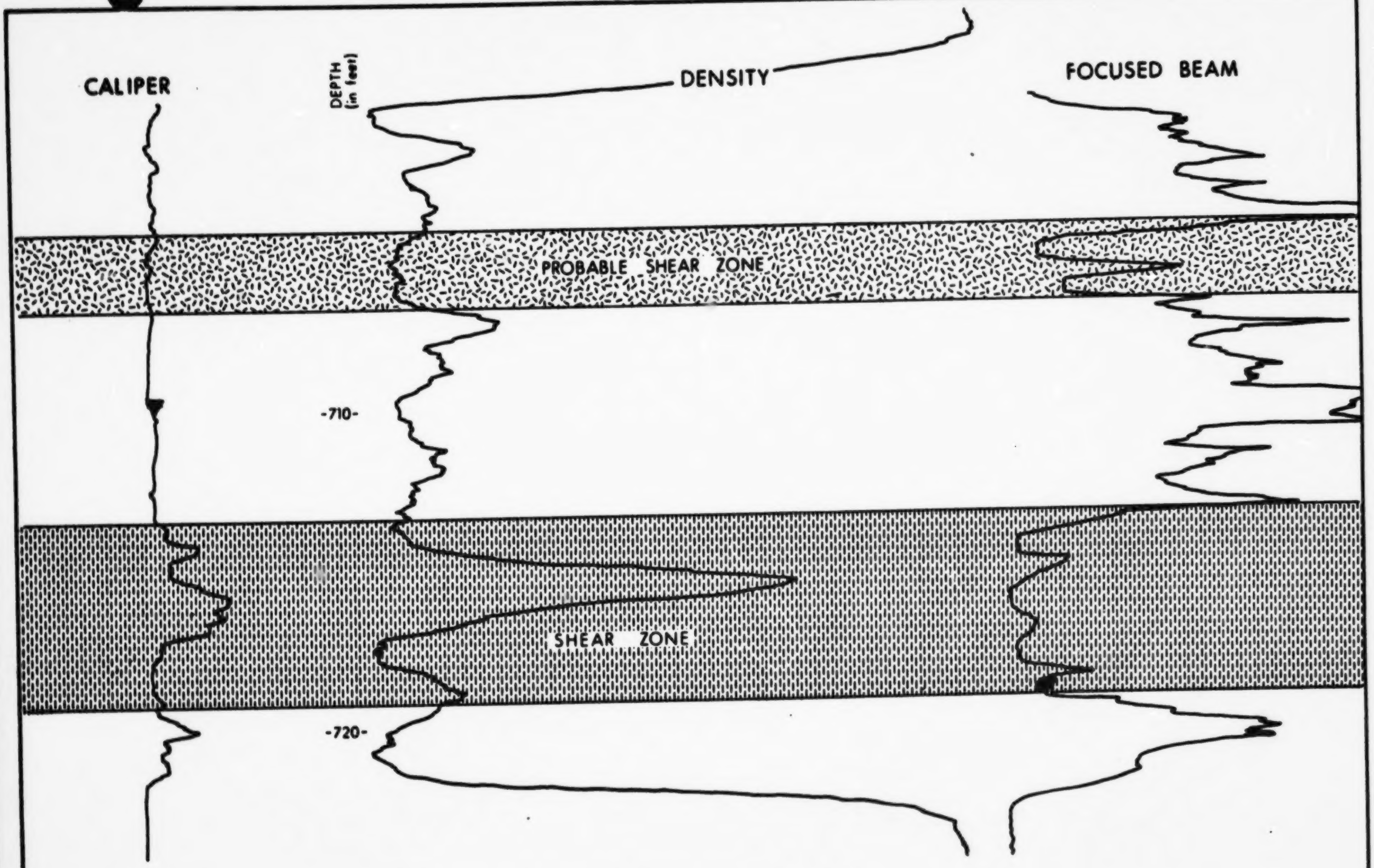
DRILLHOLE No. 81 - 12
SHEAR ZONE No. 4 SEAM

FIGURE No. 15



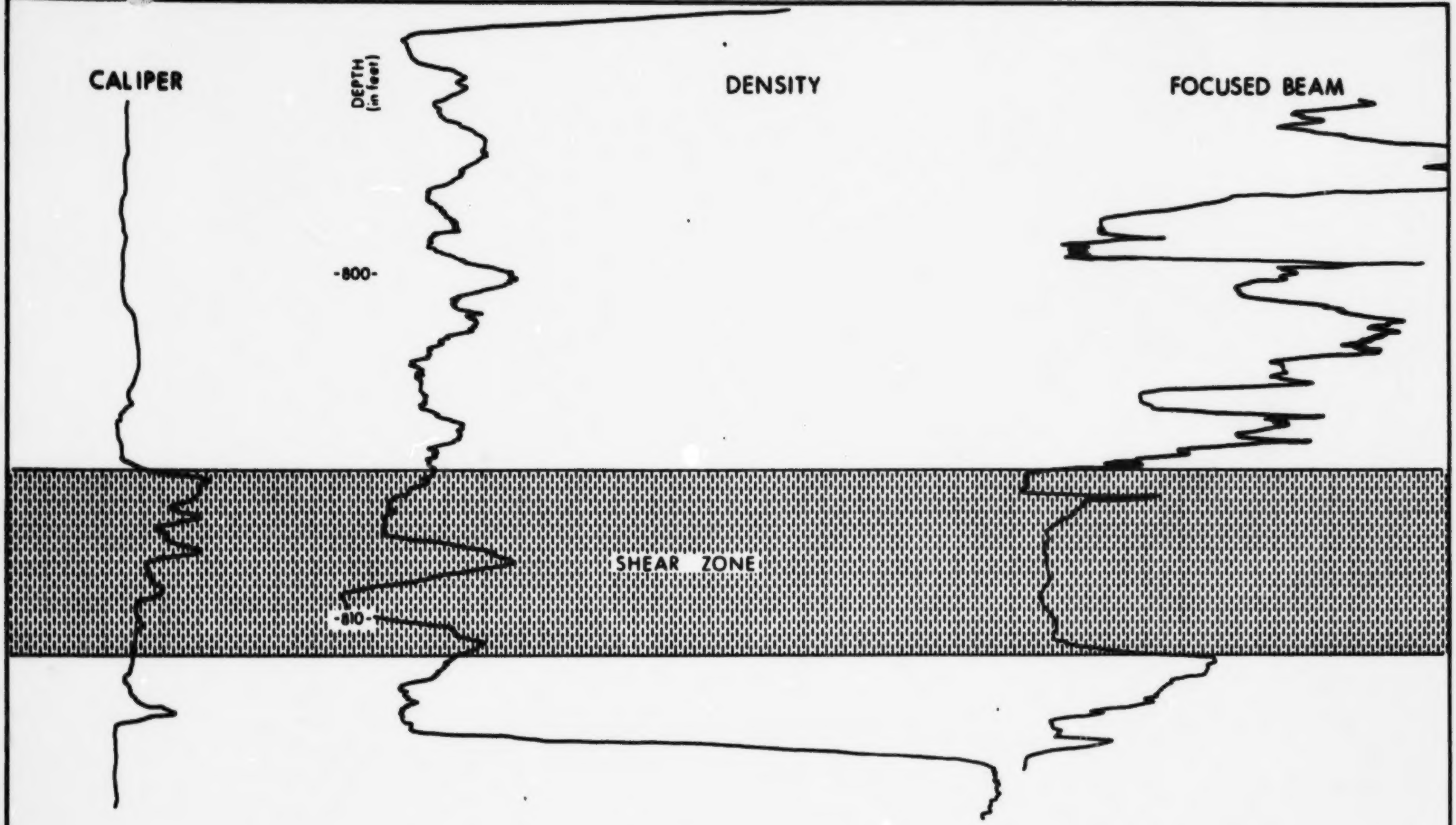
DRILLHOLE No. 81 - 13
SHEAR ZONE No. 4 SEAM

FIGURE No. 16



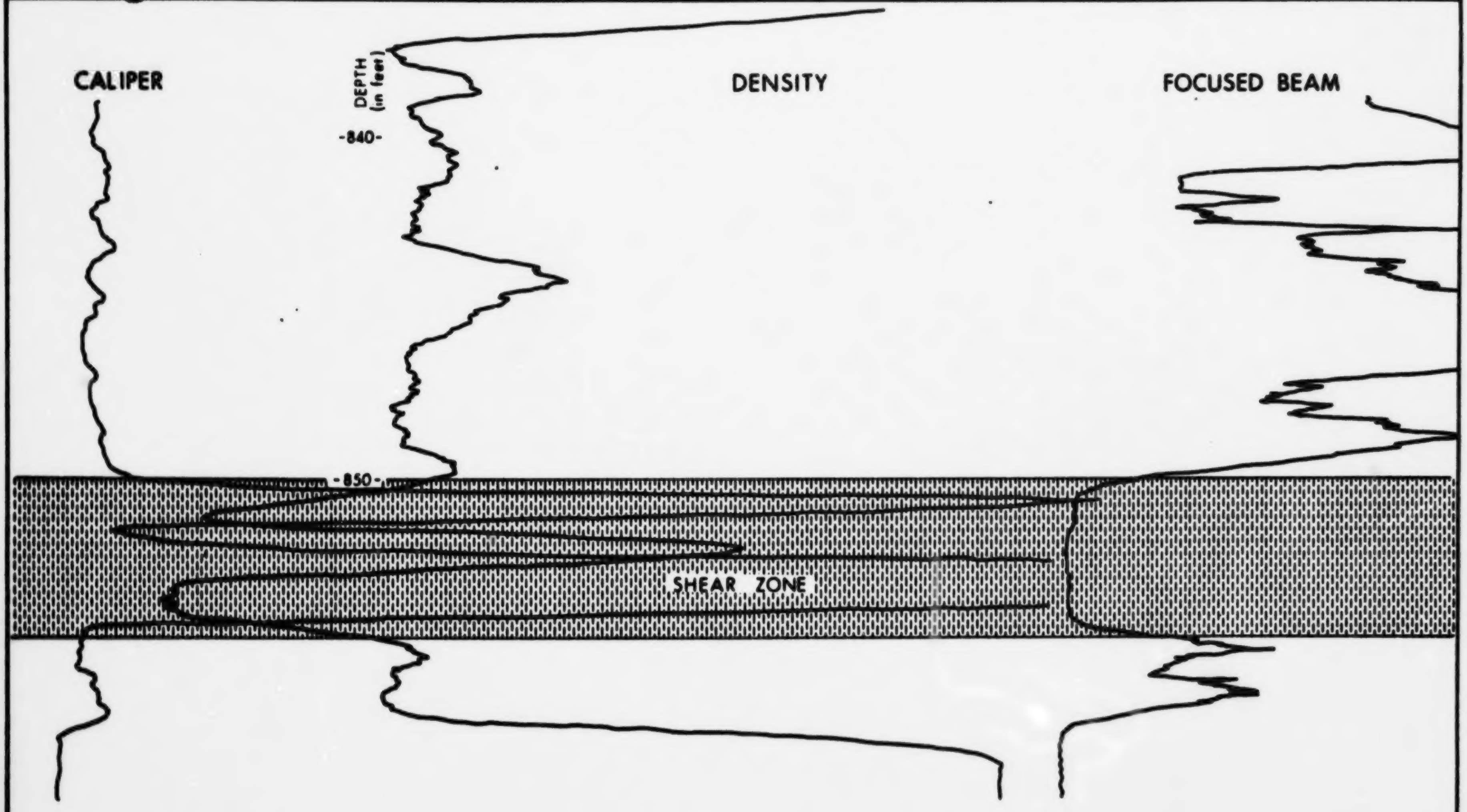
DRILLHOLE No. 81 - 14
SHEAR ZONE No. 4 SEAM

FIGURE No. 17



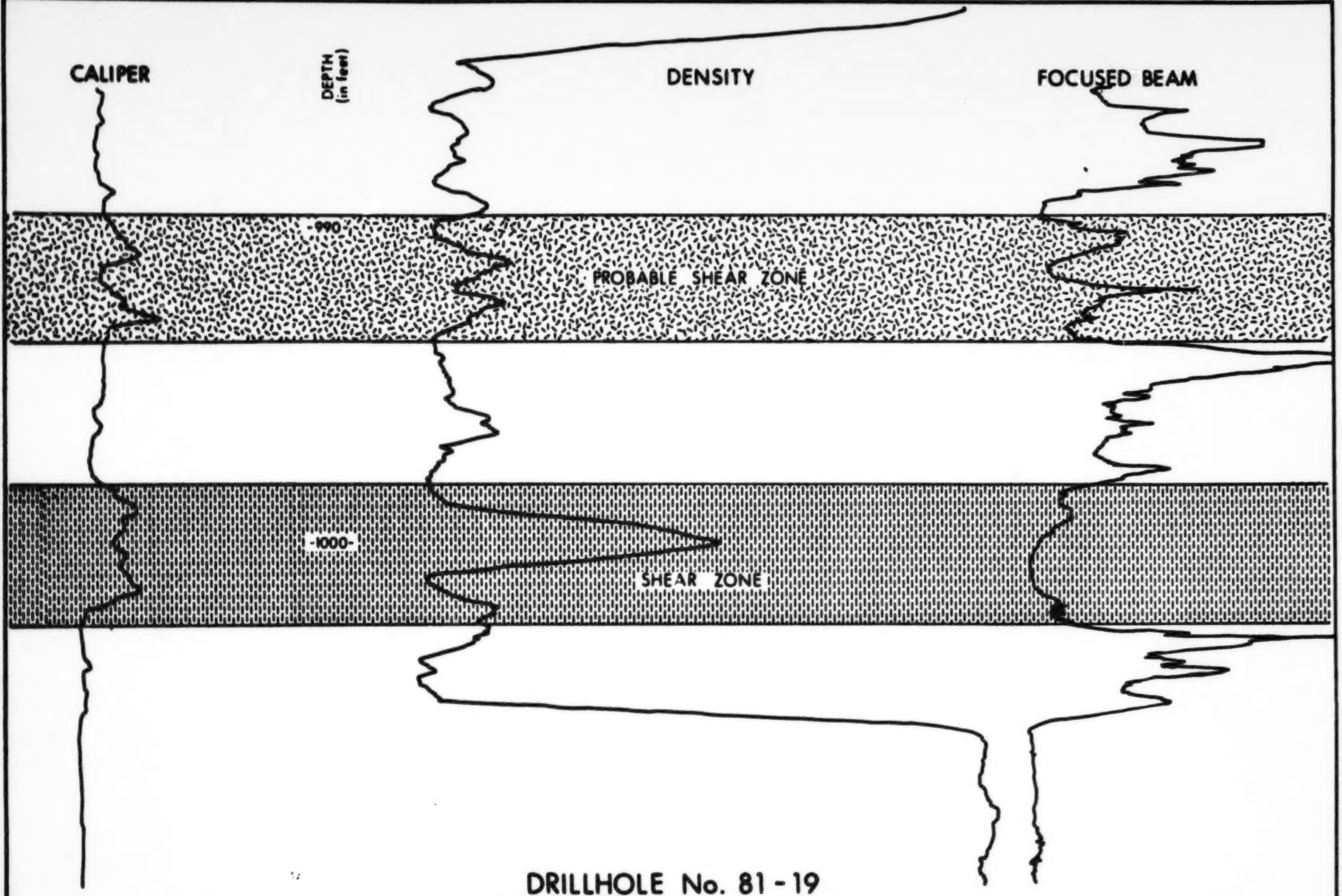
DRILLHOLE No. 81 - 16
SHEAR ZONE No. 4 SEAM

FIGURE No. 18



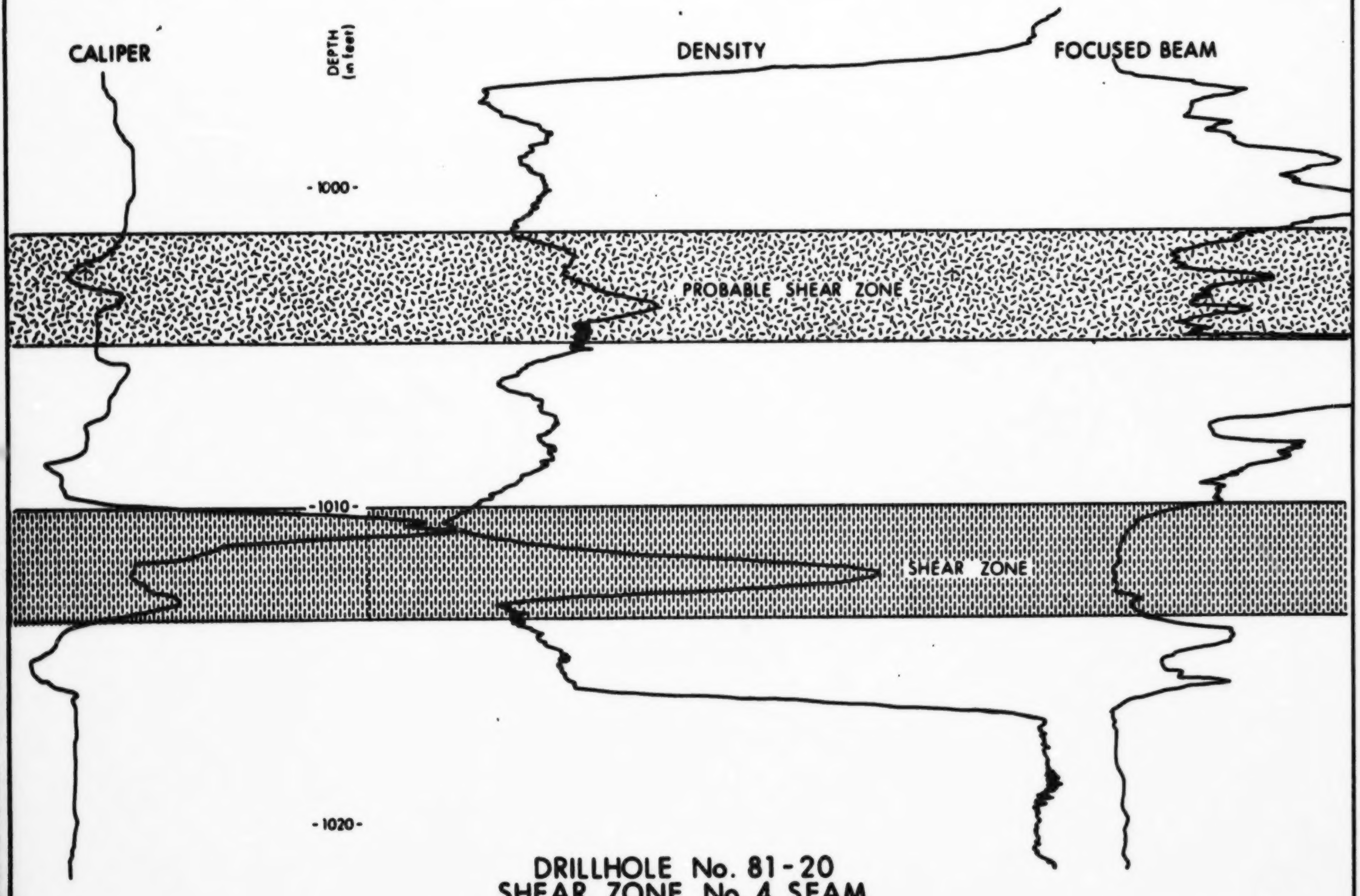
DRILLHOLE No. 81 - 17
SHEAR ZONE No. 4 SEAM

FIGURE No. 19



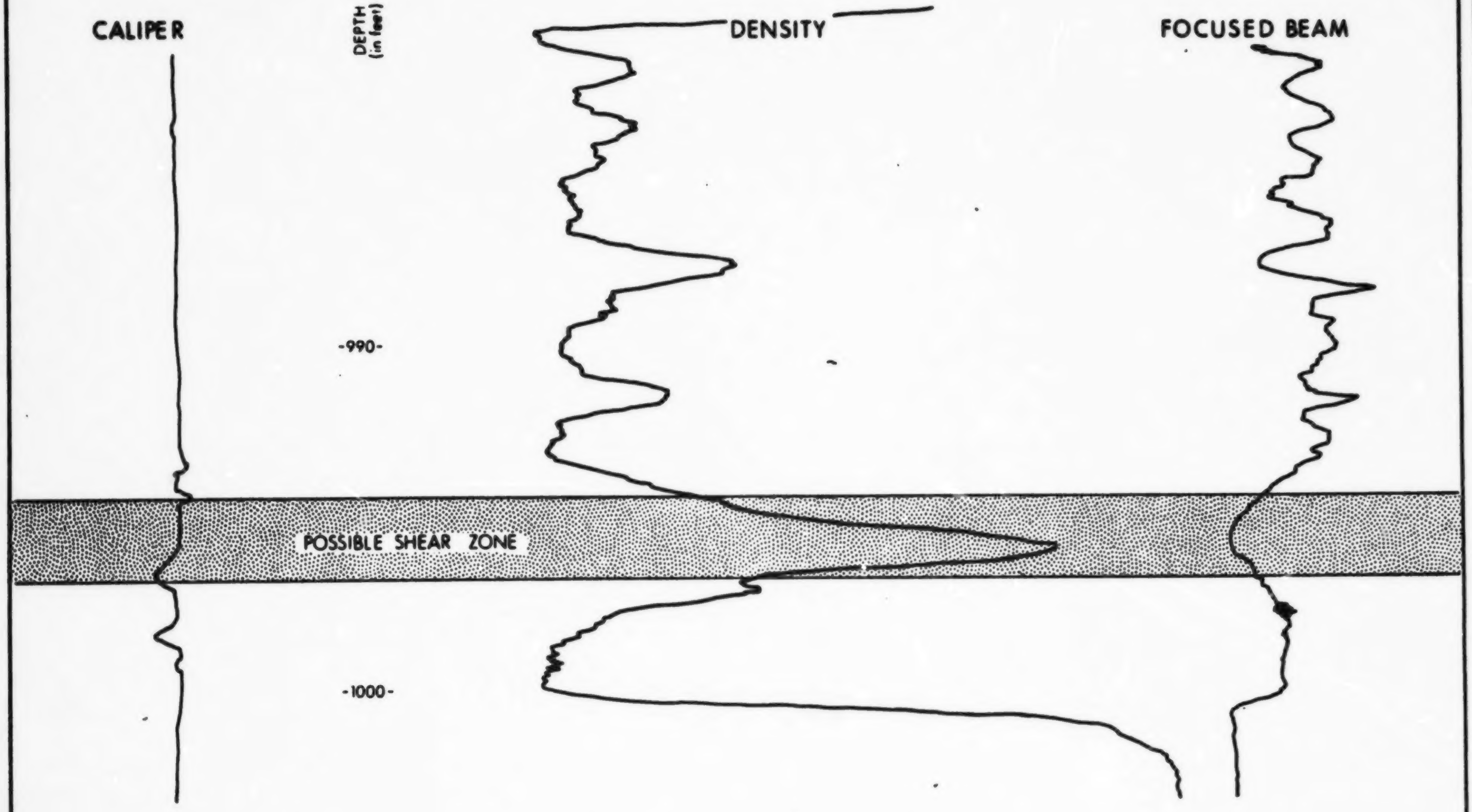
DRILLHOLE No. 81 - 19
SHEAR ZONE No. 4 SEAM

FIGURE No. 20



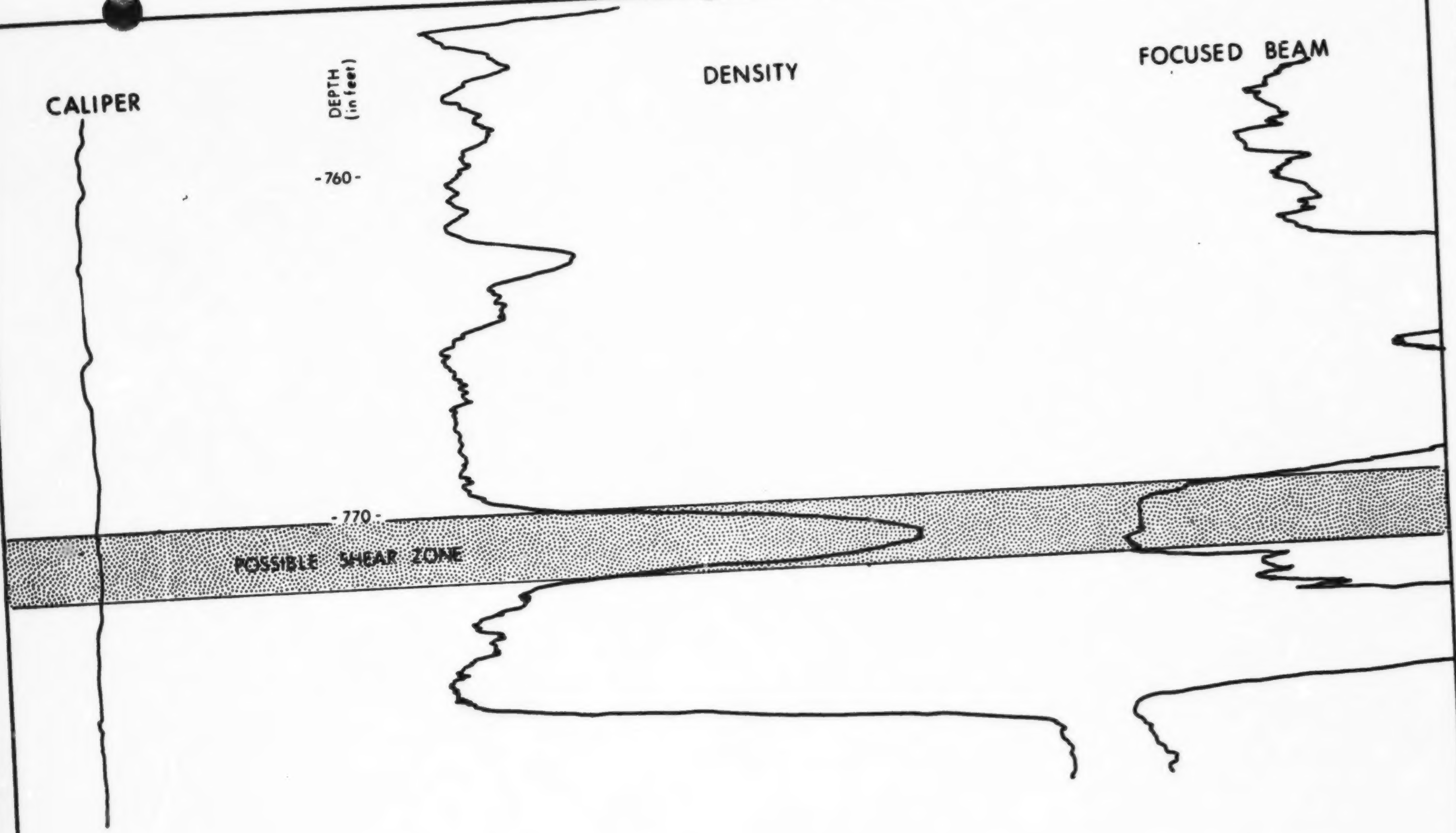
DRILLHOLE No. 81-20
SHEAR ZONE No. 4 SEAM

FIGURE No. 21



DRILLHOLE No. 81-23
SHEAR ZONE No. 4 SEAM

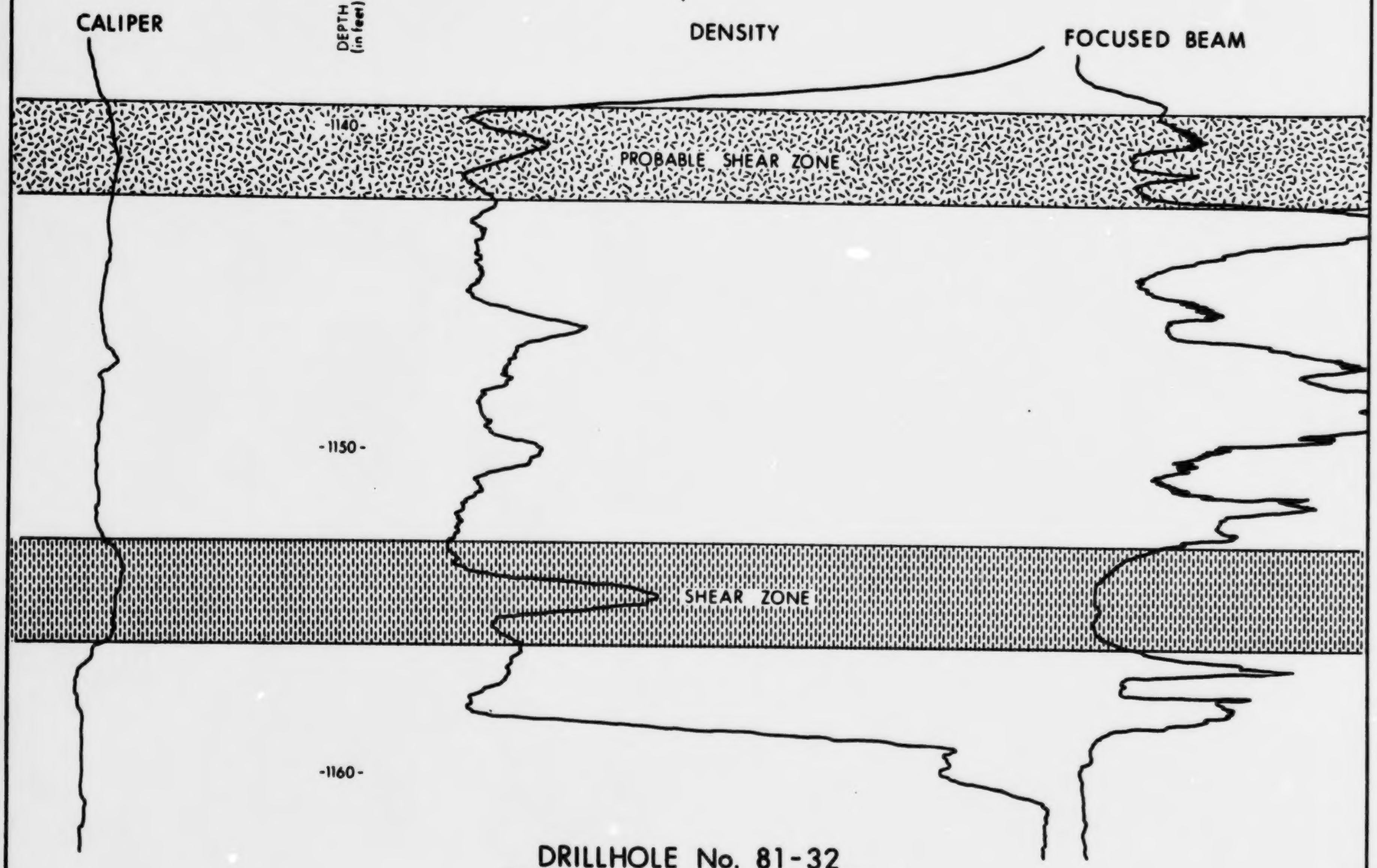
FIGURE No. 22



DRILLHOLE No. 81-24
SHEAR ZONE No. 4 SEAM

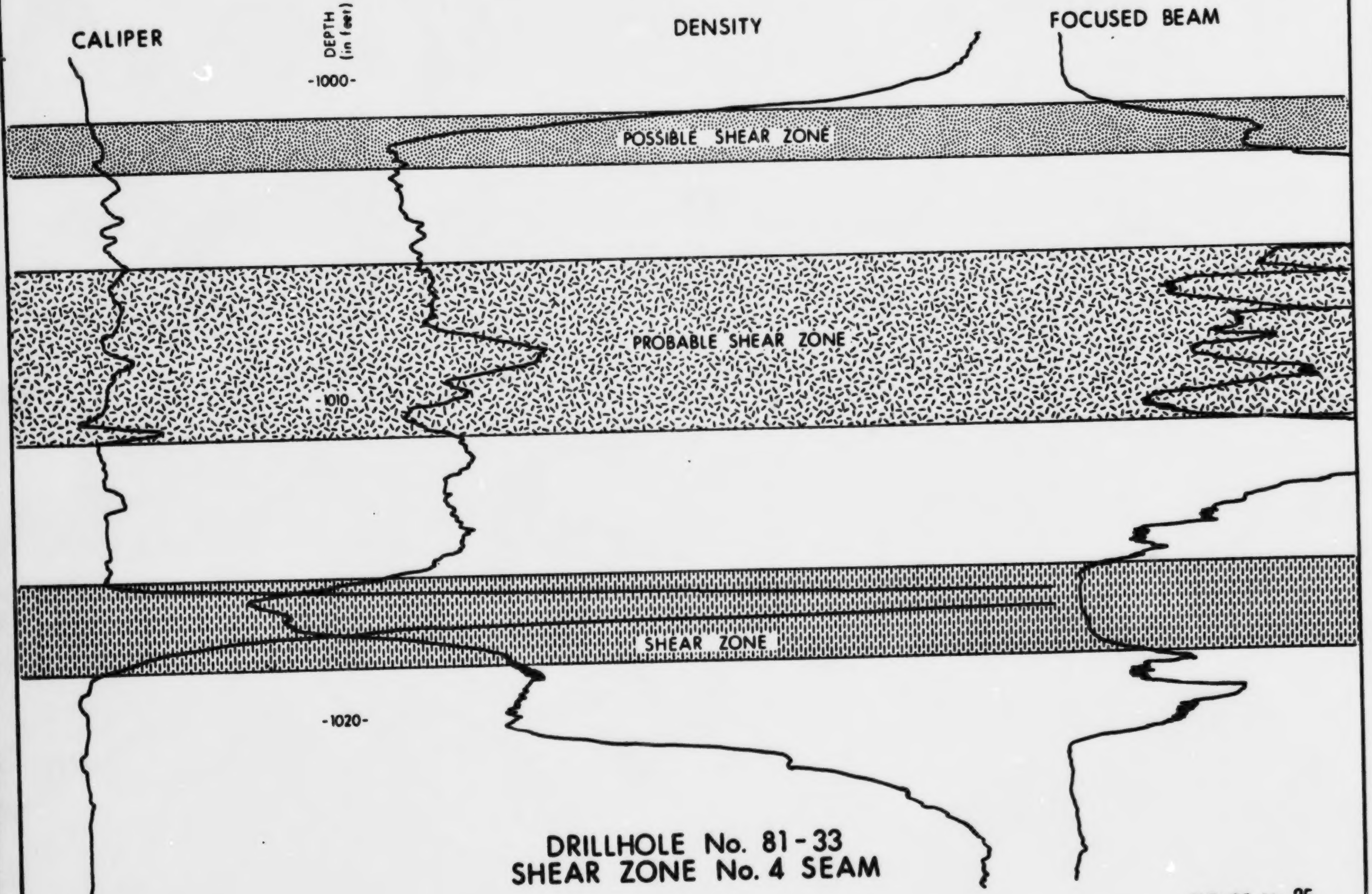
FIGURE No. 23

NorWest



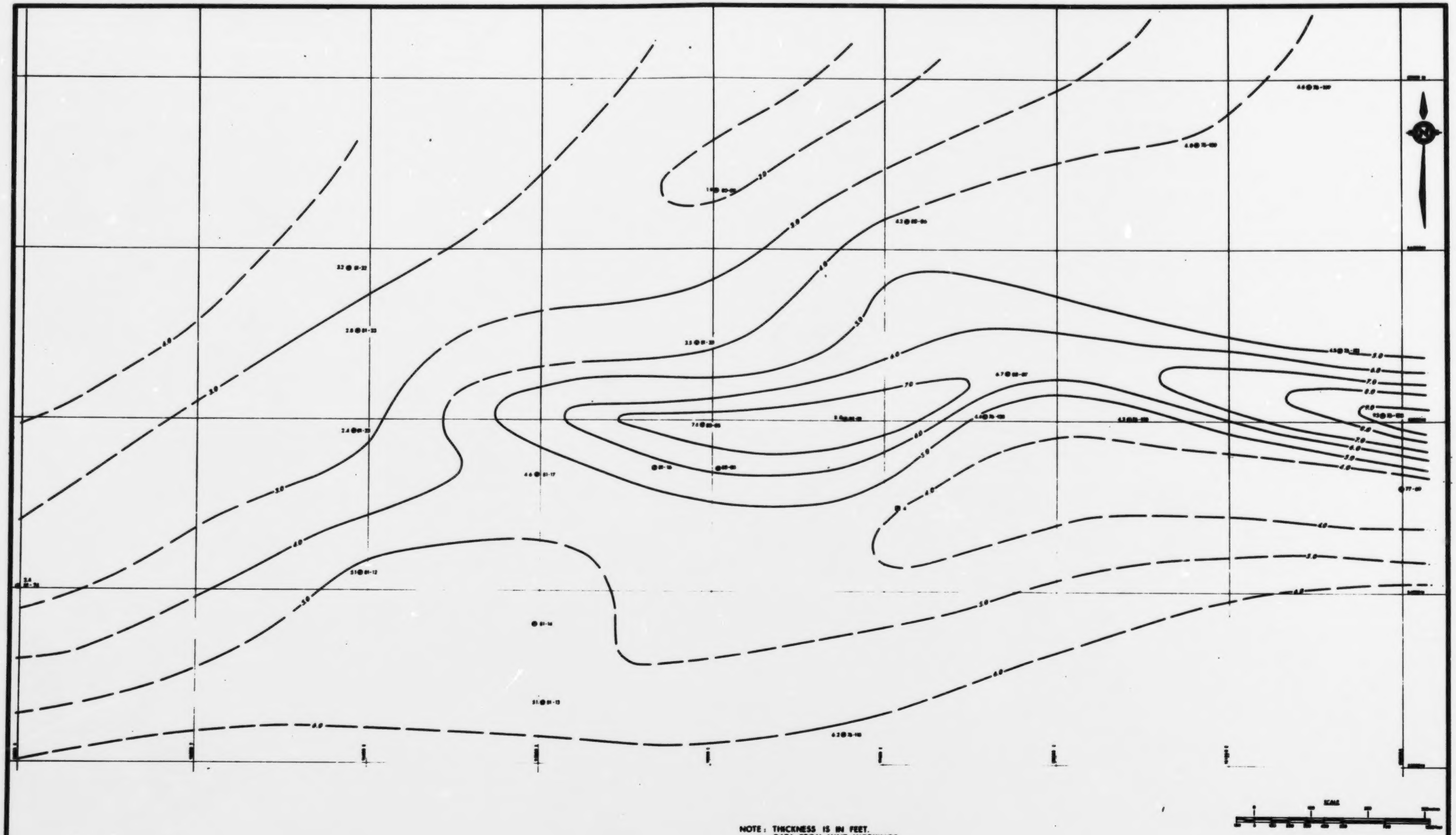
DRILLHOLE No. 81-32
SHEAR ZONE No. 4 SEAM

FIGURE No. 24



DRILLHOLE No. 81-33
SHEAR ZONE No. 4 SEAM

FIGURE No. 25

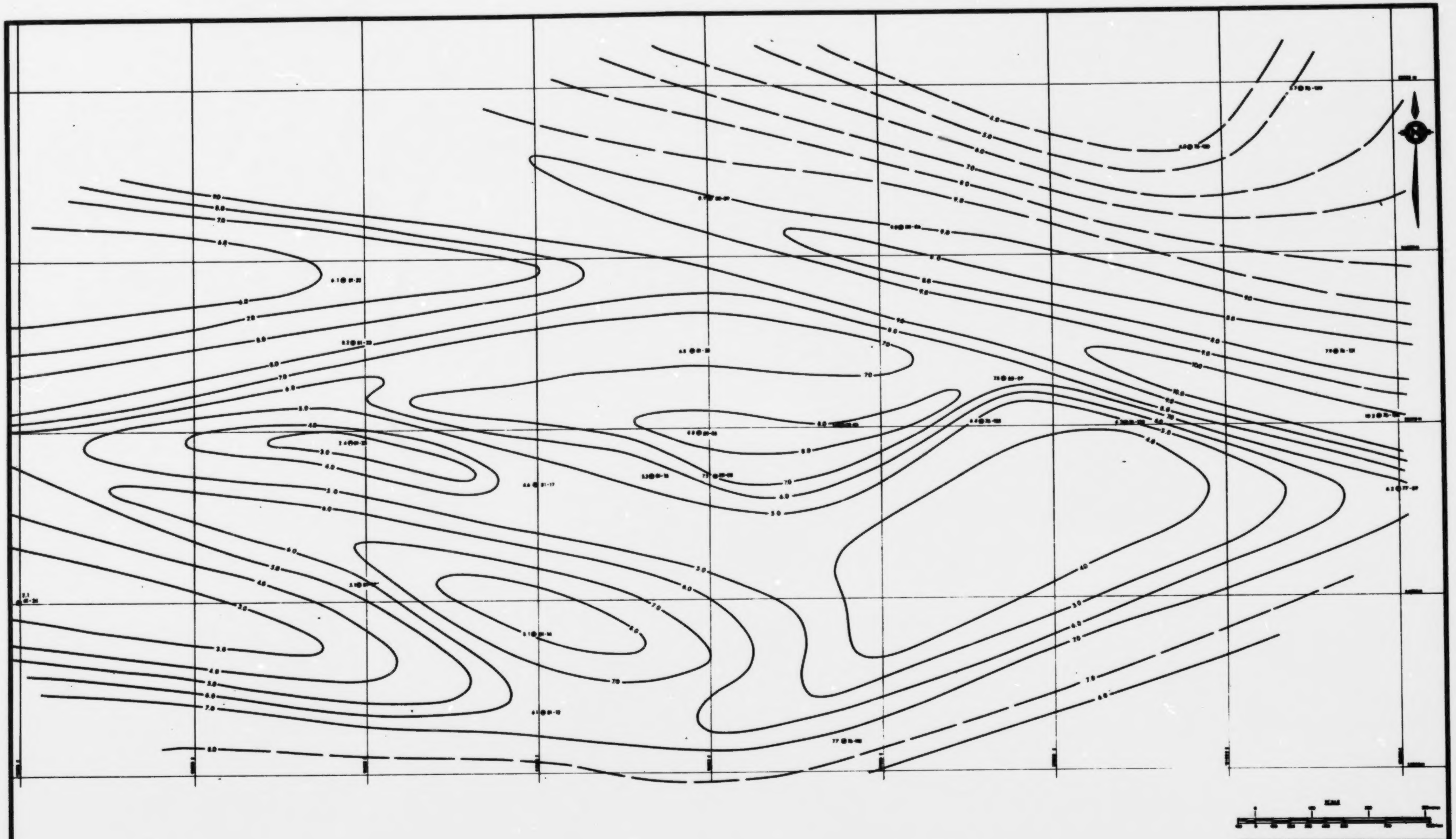


NOTE: THICKNESS IS IN FEET.
● —DATA FROM MINE WORKINGS
(LOCATION APPROXIMATE)

McINTYRE MINES LTD.

LONGWALL MINING STUDY
ISOPACH MAP
PRINCIPAL LOWER
SHEAR ZONE

H.G.S.	G.R.J.	OCT. '72
		1:200
		MAP 1

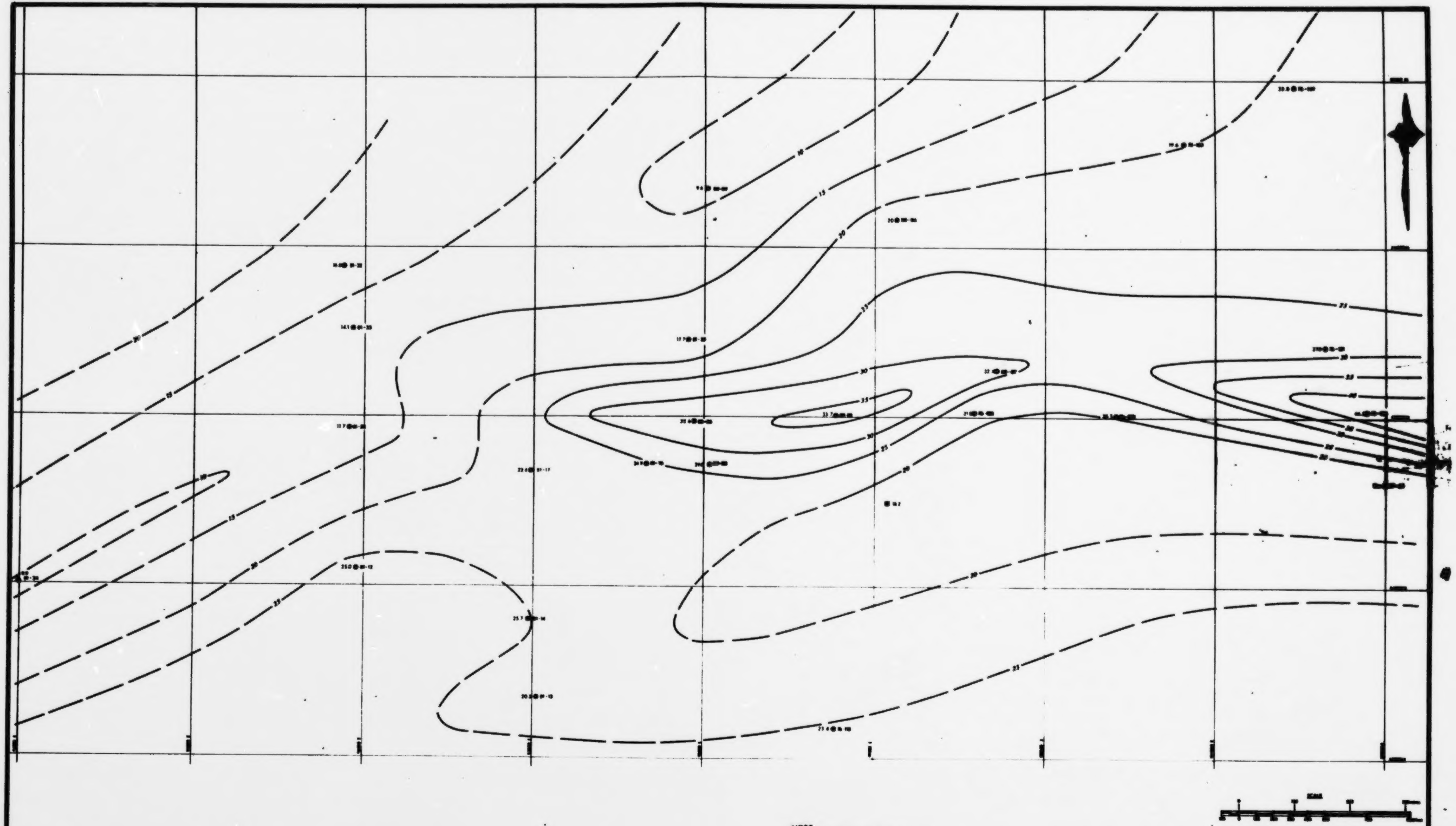


McINTYRE MINES LTD.

LONGWALL MINING STUDY

ISOPACH' MAP
TOTAL SHEAR ZONE

G.E.L.
OCT. 72
1:200
H.G.S.
MAP 2



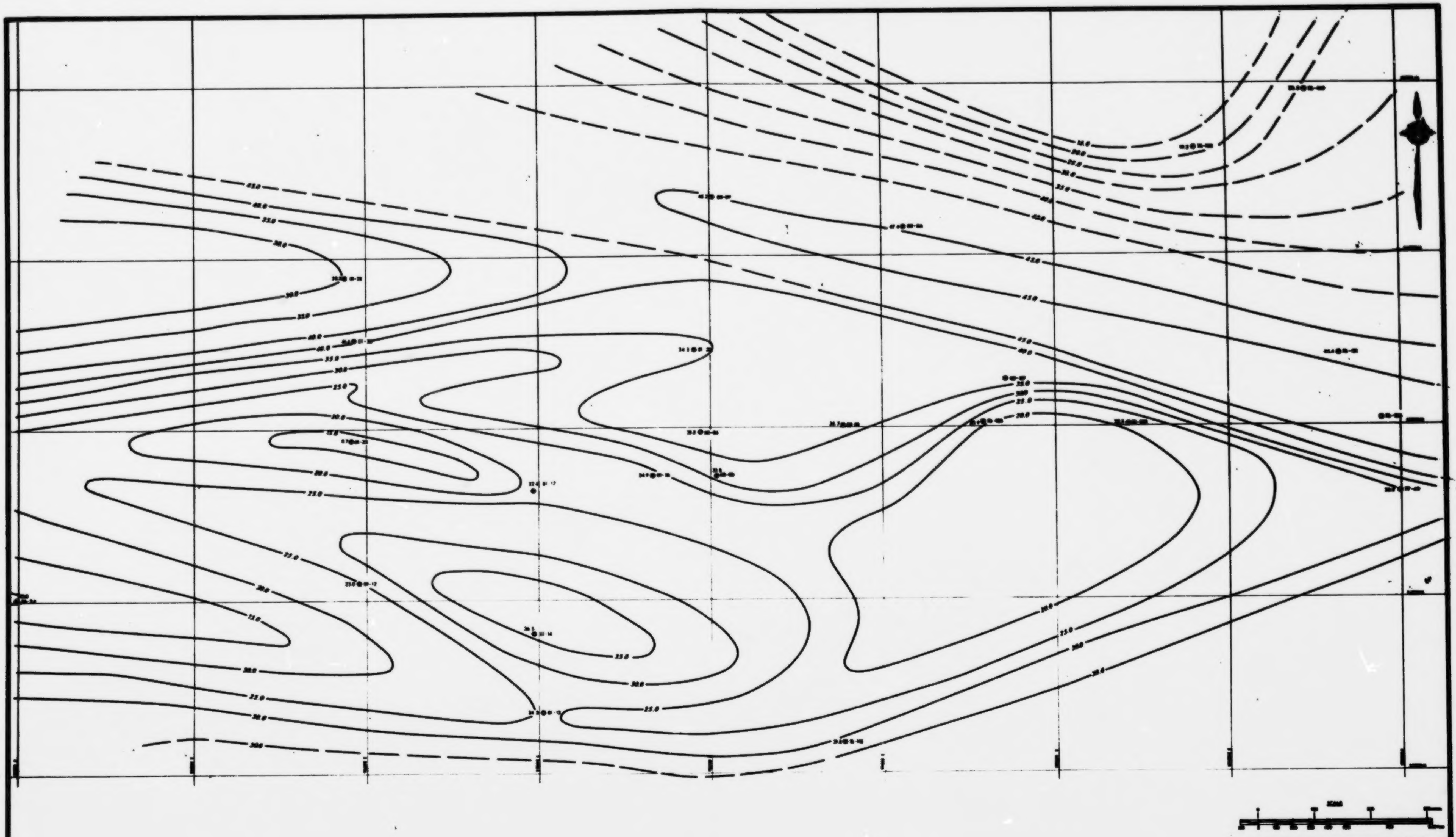
NOTE:
■ - DATA FROM MINE WORKINGS
(LOCATION APPROXIMATE)

McINTYRE MINES LTD.

LONGWALL MINING STUDY

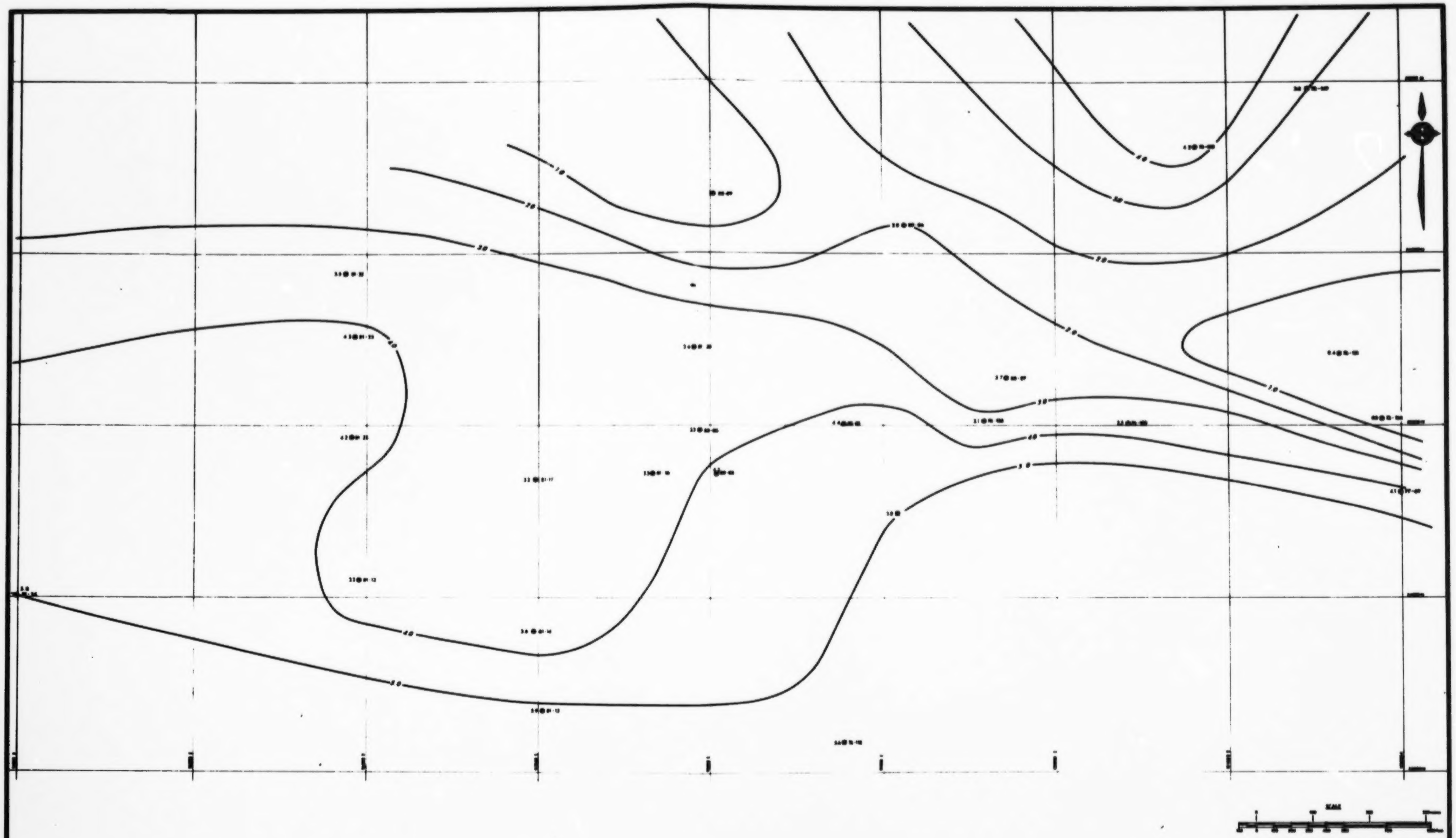
ISORATIO MAP
PRINCIPAL LOWER SHEAR ZONE
vs TOTAL SEAM THICKNESS

MAP-3.3



McINTYRE MINES LTD.

LONGWALL MINING STUDY
ISO RATIO MAP
TOTAL SHEAR ZONE vs.
TOTAL SEAM THICKNESS



NOTE:
● – DATA FROM MINE WORKINGS
(LOCATION APPROXIMATE)

McINTYRE MINES LTD.

LONGWALL MINING STUDY
ISOPACH MAP
THICKNESS OF SOLID
COAL FLOOR

4	G.E.J.	OCT. '82
H.G.S.		1:2000
		MAP-S 5

POOR

QUALITY

ORIGINAL